Standard 7-22
Minimum Design Loads and Associated Criteria for Buildings and Other Structures

SUPPLEMENT 2

Chapter 1: General

1.3.1.3 Performance Based Procedures

Structural and nonstructural components and their connections designed with performance-based procedures shall be demonstrated by analysis in accordance with Section 2.3.5 or by analysis procedures supplemented by testing to provide a reliability that is generally consistent with the target reliabilities stipulated in this section.

Structural and nonstructural components subjected to dead, live, environmental, and other loads except earthquake, tsunami, and loads from extraordinary events shall be based on the target reliabilities in Table 1.3-1. Structural systems subjected to earthquake shall be based on the target reliabilities in Table 1.3-2 and 1.3-3. The design of structures subjected to tsunami loads shall be based on the target reliabilities in Table 1.3-4. Structures, components, and systems that are designed for extraordinary events, using the requirements of Section 2.5 for scenarios approved by the Authority Having Jurisdiction, shall be based on the target reliabilities in Table 1.3-5. The analysis procedures used shall account for uncertainties in loading and resistance.

Chapter 2: Combinations of Loads

2.2 SYMBOLS

A_k = Load or load effect arising from extraordinary event A

D = Dead load

D_i = Weight of ice

E = Earthquake load

F = Load caused by fluids with well-defined pressures and maximum heights

F_a = Flood load as defined by Section 5.5

H = Load due to lateral earth pressure (including lateral earth pressure from fixed or moving surcharge loads), ground water pressure, or pressure of bulk materials

L = Live load

L_r = Roof live load

N = Notional load for structural integrity, Section 1.4

R = Rain load

S = Snow load

T = Cumulative effect of self-straining forces and effects arising from contraction or expansion resulting from environmental or operational temperature changes, shrinkage, moisture changes, creep in component materials, movement caused by differential settlement, or combinations thereof

W = Wind load

W_i = Wind-on-ice determined in accordance with Chapter 10

2.3.2 Load Combinations Including Flood Load.
When a structure is located in a flood hazard area (Section 5.3.1), the following load combinations shall be considered in addition to the basic combinations in Section 2.3.1:

4b. $1.2D + 1.0W + 1.0F_a + 1.0L + (0.5L_r \text{ or } 0.3S \text{ or } 0.5R)$

5b. $0.9D + 0.5W + 1.0F_a$

Exception:

1. The load factor on L in the combination 4b is permitted to equal 0.5 for all occupancies in which L_o in Chapter 4, Table 4.3-1 is less than or equal to 100 psf (4.78 kN/m^2), with the exception of garages or areas occupied as places of public assembly.
2. In the load combination 4b, the companion load S shall be taken as either the flat roof snow load (p_f) or the sloped roof snow load (p_s).

2.4.2 Load Combinations Including Flood Load.

When a structure is located in a flood hazard area, the following load combinations shall be considered in addition to the basic combinations in Section 2.4.1:

5b. $D + 0.6W + 0. 7F_a$

6b. $D + 0.75L + 0.75(0.6W) + 0.75(L_r \text{ or } 0.7S \text{ or } R) + 0.7F_a$

7b. $0.6D + 0.6W + 0.7F_a$

EXCEPTION:

In the load combination 6b, the companion load S shall be taken as either the flat roof snow load (p_f) or the sloped roof snow load (p_s).

Chapter 5: Flood Loads

5.1 GENERAL

The provisions of this chapter apply to buildings and other structures located in a Flood Hazard Area.

This chapter shall not apply to the design of levees, dikes, piers, wharves, roads, or bridges.

5.2 DEFINITIONS AND SYMBOLS

5.2.1 Definitions

APPROVED: Acceptable to the Authority Having Jurisdiction.

BASE FLOOD: Flood having a 1 percent chance of being equaled or exceeded in any given year (100-year flood).

BASE FLOOD ELEVATION (BFE): Elevation of flooding, including wave height, having a 1 percent chance of being equaled or exceeded in any given year.

BREAKAWAY WALL: Any type of wall subject to flooding that is not required to provide structural support to a building or other structure and that is designed and constructed such that it will collapse under specific lateral loads, defined in Section 5.3.10, and it allows for the free passage of floodwaters without damaging the supporting structure or foundation system.

DEBRIS IMPACT FORCE: Force imparted by a flood-borne debris strike.

DESIGN FLOOD:

The flood corresponding to the design mean recurrence interval assigned by risk category in accordance with Table 5.3-1, including relative sea level change.

DESIGN STILLWATER FLOOD DEPTH: The stillwater depth, above the eroded grade at an individual building or other structure, produced by the design flood.

DESIGN STILLWATER FLOOD ELEVATION: The elevation, relative to the adopted datum, of the stillwater during the design flood, including relative sea level change. See Figures 5.2-1 and 5.2-2.

ELEVATED STRUCTURE: A structure with its lowest habitable floor positioned above ground, raised on foundation walls, shear walls, posts, piers, pilings or columns, such that water may flow below the lowest habitable floor during a flooding condition.

EROSION: Wearing away of the land surface by detachment and movement of soil and rocks during a flood or storm or over a period of years, through the action of wind, water, or other geologic processes that result in a lowering of the ground surface.

FLOOD HAZARD AREA: The area subject to flooding as specified in Section 5.3.1.

FLOOD HAZARD MAP: The map delineating flood hazard areas adopted by the Authority Having Jurisdiction.

FLOOD HAZARD STUDY: Study that serves as the technical basis for a flood hazard map. A compilation and presentation of flood risk data for specific watercourses, lakes, and coastal flood hazard areas within a community.

FLOOD INSURANCE RATE MAP (FIRM): An official map of a community on which the Federal Emergency Management Agency (FEMA) has delineated both special flood hazard areas and the risk premium zones applicable to the community.

FLOOD ZONES:

A-Zone: Area subject to inundation by the 1 percent annual chance flood, (100-year flood), where wave action does not occur or where waves are less than 3 ft (0.9 m) high, designated Zone A, AE, A1- A30, A0, AH, or AR on a FIRM.

Coastal A-Zone: An area within a special flood hazard area, landward of a V-Zone or landward of an open coast without mapped Coastal High Hazard Areas. In a Coastal A-Zone, the principal source of flooding is astronomical tides and storm surges, but not riverine flooding, and the potential exists for breaking wave heights greater than or equal to 1.5 ft (0.46 m) during the base flood. The inland limit of the Coastal A-Zone is (1) the Limit of Moderate Wave Action if delineated on a FIRM, or (2) designated by the Authority Having Jurisdiction.

Coastal High Hazard Area (V-Zone): An area within a Special Flood Hazard Area, extending from offshore to the inland limit of a primary frontal dune along an open coast, and any other area that is subject to high velocity wave action from storms during the base flood. This area is designated on flood insurance rate maps (FIRMs) as V, VE, VO, or V1-30.

Shaded X-Zone: An area landward of the Special Flood Hazard Area, delineating the 0.2 percent annual chance of being equaled or exceeded in any given year flood (i.e., subject to inundation by the 500-year flood). This area is designated on FIRMs as X with a shading.

X-Zone: Areas outside the Special Flood Hazard Area. Unshaded Zone X designated areas are where the annual probability of flooding is less than 0.2 percent.

HYDRODYNAMIC LOAD: Load imposed by water flowing against and around an object or structural element.

HYDROSTATIC LOAD: Load imposed by a standing mass of water, including buoyancy.

LIMIT OF MODERATE WAVE ACTION (LiMWA): Line shown on FIRMs to indicate the inland limit of the 1.5 ft (0.46m) breaking wave height during the 1 percent annual chance flood (100-year flood).

MEAN RECURRENCE INTERVAL (MRI): The reciprocal of the annual probability of exceedance. The average time, in years, between hazard events that equal or exceed a given magnitude, based on probability and statistical analysis.

MEAN WATER LEVEL (MWL): Mean water level relative to a locally adopted datum specified on a flood hazard map in the absence of flooding.

SCOUR: The local removal of soil or sediments around an object or structural element due to an abrupt change in flow direction or velocity from the design flood.

SITE-SPECIFIC STUDY: An alternative study using analytical, numerical, or experimental methods, approved by the Authority Having Jurisdiction, to determine flood depth, associated conditions, or flood loads, for a specific location.

SPECIAL FLOOD HAZARD AREA (AREA OF SPECIAL FLOOD HAZARD): The land in the floodplain subject to a 1 percent or greater chance of flooding in any given year. These areas are delineated on a community's FIRM as A-Zones (A, AE, A1-30, A99, AR, AO, or AH) or V-Zones (V, VE, VO, or V1-30).

STILLWATER ELEVATION (SWEL): Elevation of the surface of the water in the absence of waves that is referenced to a datum, excluding relative sea level change.

WAVE HEIGHT: Vertical distance between the crest and the trough of a wave.

Controlling Wave Height: The wave height associated with the mean of the highest 2 percent of waves associated with the design flood used to determine design wave height when the waves are not depth-limited.

Design Wave Height: The wave height at an individual building or other structure associated with the design flood, used to determine scour and wave loads.

Significant Wave Height: The wave height associated with the mean of the highest one-third of waves associated with the design flood, used to determine Controlling Wave Height when not known.

WAVE LOAD: Load imparted by a wave interacting with the structure or a portion thereof.

Breaking Wave Load: Load imparted by a breaking wave, often in depth-limited conditions.

Nonbreaking Wave Load: Load imparted by waves that are not breaking, which often occurs when the wave height is less than the height associated with depth-limited conditions.

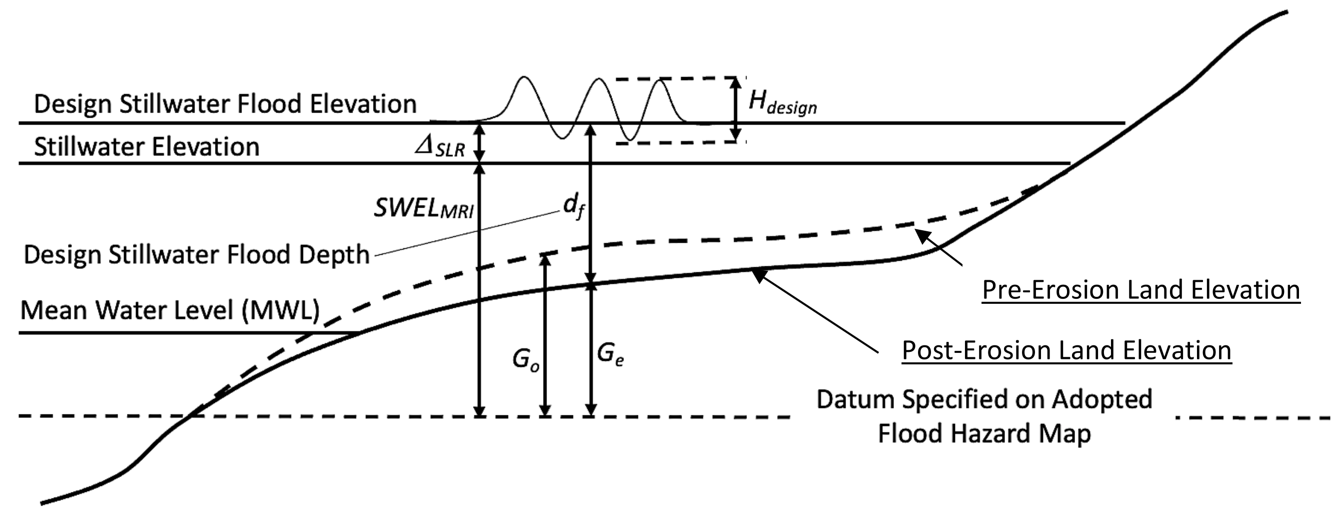


Figure 5.2-1. Flood Parameters in Coastal Areas.

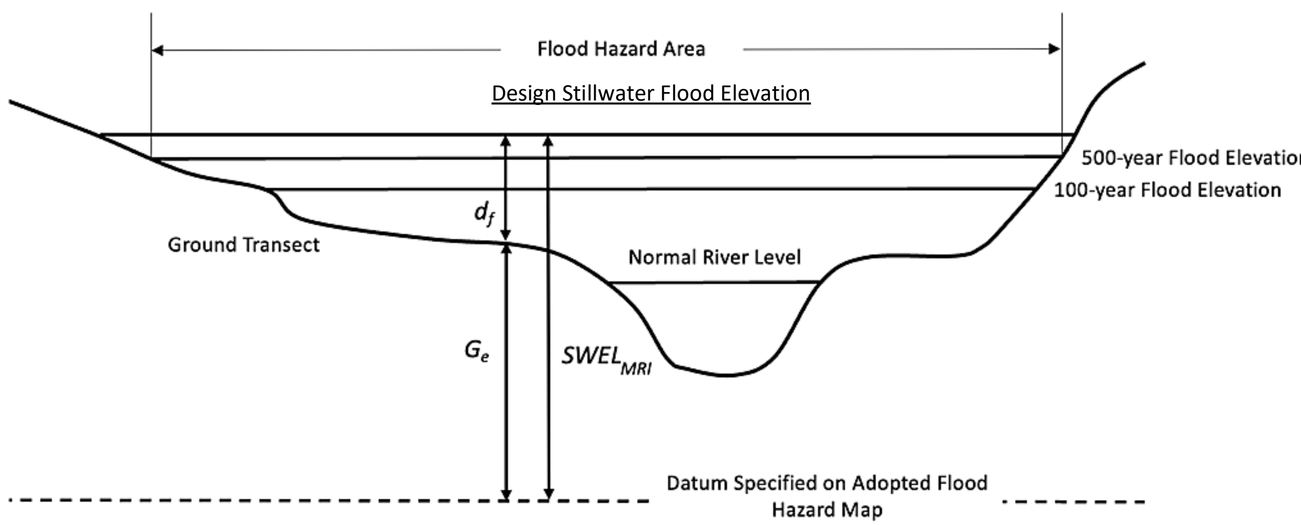


Figure 5.2-2. Flood Parameters in Riverine Areas.

5.2.2 Symbols and Notation

a = Air gap between lowest supporting horizontal structural member of lowest above grade floor and the design stillwater flood elevation, in ft (m)

A = Projected area in the flow direction exposed to moving water, including debris damming, in ft^2 (m^2)

b = Column width or width of a vertical wall, perpendicular to the direction of flow considered, in ft (m)

B = Overall width of building perpendicular to the flow direction, in ft (m)

C_{bw} = Coefficient for breaking waves

C_{br} Wave height coefficient for depth-limited breaking

C_{cx} = Debris damming closure ratio

C_D = Wave drag coefficient

C_d = Drag coefficient for submerged objects subjected to currents

C_{HC} = Scaling factor for controlling wave height

C_M = Inertia coefficient

C_{MRI} = Flood scale factor for mean recurrence interval

C_o = Debris orientation coefficient

C_R = Debris depth coefficient

C_s = Debris velocity stagnation coefficient

C_T = Wave period coefficient

C_V = Velocity coefficient

C_{Vmax} = Coefficient of maximum velocity

D = Pile or column diameter, in ft (m)

D_{SW} = Self-weight of the structure or portion of structure being evaluated inclusive of permanent fixed elements and equipment, in lb (kN)

d_f = Design stillwater flood depth, in ft (m)

E = Modulus of elasticity, in psi (kPa)

F_a = Flood load used in the Chapter 2 load combinations, in lb (kN)

F_B = Uplift force caused by buoyancy, in lb (kN)

F_{bw} = Maximum net lateral force resulting from breaking waves on a vertical pile or column, in lb (kN)

F_{di} = Debris impact force, in lb (kN)

F_{drag} = Hydrodynamic drag force, exerted by moving water on structural components immersed in the flow and buildings immersed in the flow, in lb (kN)

F_{BNV} = Horizontal component of breaking wave force per unit length on a non-vertical wall, in lb/ft (kN/m)

F_{BOI} = Horizontal component of obliquely incident breaking wave force, in lb/ft (kN/m)

F_{BRK} = Breaking wave force per unit length on a vertical wall, in lb/ft (kN/m)

F_h = Lateral force caused by the hydrostatic pressure on one side of a vertical wall per unit width, in lb/ft (kN/m)

$F_{lateral}$ = Maximum lateral component of the flood load, F_a, in lb (kN)

F_t = Lateral wave-induced force per unit length on the vertical wall or structure, in lb/ft (kN/m)

F_m = Maximum net lateral force resulting from a nonbreaking wave, in lb (kN)

g = Acceleration due to gravity, in ft/s^2 (m/s^2)

G_e = Elevation of grade at the building or other structure inclusive of effects of erosion in ft (m)

G_o = Original grade at the building or other structure in ft (m)

H = Wave height from crest to trough, in ft (m)

H_a = Resultant force from active lateral earth pressures, in lb (kN)

H_b = Breaking wave height, in ft (m)

H_c = Controlling wave height, in ft (m)

H_{c100} = Controlling wave height for the 100-year MRI, in ft (m)

H_{cMRI} = Controlling wave height corresponding to the risk category and MRI, in ft (m)

H_{design} = Design wave height, in ft (m)

H_p = Resultant force from passive lateral earth pressures, in lb (kN)

H_s = Significant wave height, in ft (m)

h = Submerged height of column or wall above its foundation or structural floor level, in ft (m)

h_c = Height in ft (m) to the top of the vertical wall or structure above the design stillwater flood elevation

k_e = Effective stiffness of the impacting debris or the effective lateral stiffness of the impacted structural element(s) deformed by the impact, in lb/ft (kN/m).

L = Wavelength, in ft (m)

m_{debris} = Mass of the debris (W_{debris}/g), in lb s^2/ft (kg).

p_h = Hydrostatic pressure at a given depth z, in lb/ft^2 (kN/m^2)

p_1 = Pressure at the design stillwater flood elevation, in lb/ft^2 (kN/m^2)

p_2 = Pressure at the top of the vertical wall or structure, in lb/ft^2 (kN/m^2)

p_3 = Pressure at the eroded ground elevation, in lb/ft^2 (kN/m^2)

p_4 = Pressure at the base of the structure, in lb/ft^2 (kN/m^2)

p_{1B} = Breaking wave pressure acting at the design stillwater flood elevation, in lb/ft^2 (kN/m^2)

R = Vertical wave runup distance from the design stillwater flood elevation, in ft (m)

R_B = Allowable uplift resisting capacity of structural foundation elements and/or other conditions resisting uplift, in lb (kN)

R_p = Allowable lateral resisting capacity of deep foundations, external structural foundation elements and/or other conditions resisting sliding, in lb (kN)

s = Average clear spacing of column or wall to the adjacent column or wall in ft (m)

S_m = Maximum scour depth, in ft (m)

$SWEL_{MRI}$ = Stillwater elevation corresponding to the risk category and MRI, in ft (m)

$SWEL_{100}$ = Stillwater elevation for the 100-year MRI, in ft (m)

T_p = Wave period corresponding to the wave height, in sec (s)
V = Design flood velocity, in ft/s (m/s)
V_{max} = Maximum velocity of water, in ft/s (m/s)
V_w = Volume of displaced water, in ft^3 (m^3)
W = Nondimensional parameter

W_{debris} = Minimum debris weight, in lb (kN)
$W_{lateral}$ = Maximum total lateral wind load on the structure, in lb (kN)
W_{uplift} = Maximum total vertical uplift wind load on the structure, in lb (kN).

Z_{datum} = Elevation of mean water level based on local datum, in ft (m)

z = Depth below design stillwater flood elevation, in ft (m)
α_V = Vertical angle between nonvertical surface and the horizontal
α_H = Horizontal angle between the direction of wave approach and the vertical surface

$\alpha*$ = Impulsive wave pressure coefficient
ϕ_m = Force coefficient
ρ = Mass density of water, in lb s^2/ft^4 (kg/m^3)
Δ_{SLR} = Relative sea level change for coastal sites in ft (m)

γ_w = Specific weight of water, in lb/ft^3 (kN/m^3)
$\eta*$ = Height in ft (m), measured above the design stillwater flood elevation, below which the wave pressure is assumed to act, or the minimum height at which the wave pressure equals zero

μ = Coefficient of sliding friction at slip plane being considered between structure on shallow foundations and subgrade

5.3 DESIGN REQUIREMENTS

5.3.1 Flood Hazard Area

For Risk Categories II, III, and IV structures, the Flood Hazard Area shall be the 500-year floodplain designated as the Special Flood Hazard Area and the Shaded X-Zone. For Risk Category I structures, the Flood Hazard Area shall be the 100-year floodplain designated as the Special Flood Hazard Area.

5.3.2 Design Loads

Structural systems of buildings or other structures located within the Flood Hazard Area shall be designed, constructed, connected, and anchored to resist the loads associated with the design flood as defined in this chapter.

5.3.3 Design Stillwater Flood Depth

The design stillwater flood depth, d_f, in ft (m) shall be determined in accordance with Equation (5.3-1):

$$d_f = (SWEL_{MRI} - G_e) + \Delta_{SLR} \qquad (5.3\text{-}1)$$

where

$SWEL_{MRI}$ = Stillwater elevation corresponding to the risk category and MRI defined in Table 5.3-1 provided by a flood hazard study adopted by the Authority Having Jurisdiction, in ft (m). Where the stillwater elevation for a given MRI is not provided in the flood hazard study, the 100-year stillwater elevation shall be scaled to the required MRI per Section 5.3.3.1,

G_e = Elevation of grade at the building or other structure inclusive of effects of erosion in ft (m), per Section 5.3.5, and

Δ_{SLR} = Relative sea level change for coastal sites in ft (m); see Section 5.3.4. Δ_{SLR} shall not be taken as less than 0.

5.3.3.1 Stillwater Elevation Determination When MRI Data Not Available

Where MRI data is not available, $SWEL_{MRI}$ shall be determined according to Equation (5.3-2):

$$SWEL_{MRI} = C_{MRI} (SWEL_{100} - Z_{datum}) + Z_{datum} \qquad (5.3\text{-}2)$$

where

$SWEL_{100}$ = Stillwater elevation for the 100-year MRI provided by a flood hazard study adopted by the Authority Having Jurisdiction in ft (m),

C_{MRI} = Flood scale factor associated with the MRI from Table 5.3-1 for different locations, and

Z_{datum} = Elevation of mean water level based on local datum, in ft (m). For riverine sites, Z_{datum} shall be taken as the annual high-water level. Z_{datum} shall be permitted to be taken as zero for coastal sites. Values for $SWEL_{100}$, $SWEL_{MRI}$, and G_e shall all reference the same local datum.

Table 5.3-1. Design Flood MRI Scaling Factors.

Risk Category	MRI (year)	Annual Exceedance Probability (AEP)	C_{MRI} Gulf of Mexico Coastal Sites[1]	C_{MRI} All Other Coastal Sites[1]	C_{MRI} Great Lakes Sites[2]	C_{MRI} Riverine Sites
I	100	1.00%	1.00	1.00	1.00	1.00
II	500	0.20%	1.35	1.25	1.15	1.35
III	750	0.13%	1.45	1.35	1.20	1.45
IV	1,000	0.10%	1.50	1.40	1.25	1.50

[1]Gulf Coast site scale factors are for coastlines of Texas, Louisiana, Mississippi, Alabama, and Florida west of 80.75 degrees W. All other coastlines shall be taken as Other.

[2]If flood loading is being considered on other lakes, the scale factors for riverine sites shall be used.

5.3.4 Effects of Relative Sea Level Change

The effects of relative sea level change shall be included in the calculation of flood conditions and flood loads for sites whose flooding comes from coastal sources. A project lifecycle of not less than 50 years shall be used for this quantification. The minimum rate of relative sea level change shall be the historically recorded sea level change rate for the site over a 50-year period. The increase in relative sea level during the project lifecycle of the structure shall be added to the design stillwater flood elevation as required by Section 5.3.3.

5.3.5 Erosion

The effects of erosion shall be included in the calculation of flood conditions and flood loads. The effects of erosion need not exceed the depth of non-erodible strata.

5.3.6 Flood Velocity

5.3.6.1 Flood Velocity in Coastal Areas

For coastal areas, the velocity of water V in the absence of neighboring structures shall be obtained by one of the following three methods: (1) by using Equation (5.3-4), (2) by numerical modeling, or (3) by laboratory testing (physical modeling). When Method 2 or 3 are used, design flood parameters shall be determined using site-specific studies in accordance with Section 5.3.11.

$$V = C_V \sqrt{g\, d_f} \qquad (5.3\text{-}4)$$

where

V = Design flood velocity, in ft/s (m/s),

g = Acceleration due to gravity, 32.2 ft/s^2 (9.81 m/s^2),

d_f = Design stillwater flood depth, in ft (m), and

C_V = Velocity coefficient, taken as 0.5.

The maximum velocity of water, V_{max}, for coastal areas need not be greater than C_{VMAX} *10 ft/s (C_{VMAX} * 3.05 m/s), where C_{VMAX} is the coefficient obtained from Table 5.3-2 used to scale to the maximum velocity.

Table 5.3-2. Design Flood MRI and Scaling Factors for Maximum Velocity.

Risk Category	All Coastal Sites C_{VMAX}
I	1.00
II	1.35
III	1.45
IV	1.50

5.3.6.2 Flood Velocity in Riverine Areas

For riverine areas, the average velocity of water V shall be obtained by one of the following four methods: (1) from a flood hazard study, (2) by analytical methods using open channel flow hydraulics, (3) by numerical modeling, or (4) by laboratory testing (physical modeling). When Method 2, 3 or 4 are used, design flood parameters shall be determined using the site-specific hazard procedures of Section 5.3.11.

5.3.7 Wave Effects

The effects of waves shall be included for both V-Zones and A-Zones. In areas subjected to riverine flooding only, the effects of waves are permitted to be neglected.

5.3.7.1 Wave Height

The design wave height H_{design} at the site in ft (m) shall be obtained by one of the following four methods: (1) by assuming depth-limited breaking wave conditions, (2) from a flood hazard study, (3) by numerical modeling, or (4) by laboratory testing (physical modeling). When Method 3 or 4 are used, design flood parameters shall be determined using the site-specific hazard procedures of Section 5.3.11.

If Method 1 is used, the wave shall be considered a breaking wave for the calculation of scour depths in Section 5.3.8 and wave loads in Section 5.4.4. For Method 1, the design wave height H_{design} shall equal the breaking wave height H_b in ft (m) and shall be determined by

$$H_{design} = H_b \qquad (5.3\text{-}5)$$

where H_b is calculated as follows:

$$H_b = C_{br} d_f \qquad (5.3\text{-}6)$$

where

d_f = Design stillwater flood depth, in ft (m), and

C_{br} = Wave height coefficient for depth-limited breaking taken as 0.78.

If Method 2, 3, or 4 is used, the design wave height H_{design} in ft (m) for the calculation of scour depths in Section 5.3.8 and wave loads in Section 5.4.4 shall be determined by

$$H_{design} = H_c \qquad (5.3\text{-}7)$$

where H_c is calculated as follows:

H_c = Controlling wave height in ft (m) from Method 2, 3, or 4, taken as

$$H_c = 1.6\, H_s \qquad (5.3\text{-}8)$$

where H_s = Significant wave height.

When wave heights are not known at the site, but are known at the shoreline, the wave heights are permitted to be transformed to the site accounting for obstructions and depth limitations.

If the controlling wave height is specified by a 100-year design flood, then the controlling wave height shall be adjusted to the controlling wave height corresponding to the MRI design flood event using Table 5.3-3 such that $H_{cMRI} = C_{HC} * H_{c100}$.

If H_{design} at the site is equal to or exceeds H_b from Eq. (5.3-6), then the wave shall be breaking and $H_{design} = H_b$.

Table 5.3-3 Design Flood MRI and Scaling Factors for Controlling Wave Height.

Risk Category	All Coastal Sites C_{HC}
I	1.00
II	1.30
III	1.35
IV	1.40

5.3.7.2 Wave Period and Wavelength.

The wave period T_p in sec (s) corresponding to the wave height shall be obtained by one of the following four methods: (1) by using Equation (5.3-9), (2) from a flood hazard study, (3) by numerical modeling, or (4) by laboratory testing (physical modeling). When Methods 3 or 4 are used, design flood parameters shall be determined using site-specific studies in accordance with Section 5.3.11.

$$T_p = C_T \sqrt{\frac{H_{design}}{g}} \tag{5.3-9}$$

where

T_p = Wave period corresponding to the wave height, in sec (s),

C_T = Wave period coefficient equal to 12.1, and

g = Acceleration due to gravity, 32.2 ft/s^2 (9.81 m/s^2).

The wavelength, L, in ft (m) shall be calculated by

$$L = \frac{gT_p^{\,2}}{2\pi}\left(1 - e^{-\left(\frac{2\pi}{T_p}\sqrt{\frac{d_f}{g}}\right)^{\frac{5}{2}}}\right)^{\frac{2}{5}} \tag{5.3-10}$$

5.3.8 Scour

Scour shall be calculated by using either the procedures outlined in Sections 5.3.8.1 and 5.3.8.2. Scour shall be considered for surfaces subject to hydrodynamic forces above the non-erodible strata.

EXCEPTION: Analysis of scour is not required if soils adjacent to structural foundations are non-erodible or protected against scour by structures designed for anticipated flood loads.

5.3.8.1 Scour at Walls

Scour depth at the exposed face of walls shall be calculated in accordance with Section 5.3.8.1.1, for scour due to nonbreaking waves, or in accordance with Section 5.3.8.1.2, for scour due to breaking waves. Waves shall be evaluated per Section 5.3.7. A structural element with a ratio of the design stillwater flood depth to the lateral dimension facing the wave less than three shall be considered to act as a wall. Tightly spaced piles or columns where the clear distance between piles or columns is less than one half of the lateral dimension of an individual pile or column facing the wave shall be considered to act as a wall.

5.3.8.1.1 Scour Due to Nonbreaking Waves

The maximum scour depth S_m in ft (m) at walls for nonbreaking waves shall be calculated by Equation (5.3-11):

$$S_m = \frac{0.25\, H_{design}}{\left[\sinh\left(\frac{2\pi d_f}{L}\right)\right]^{1.35}} \quad (5.3\text{-}11)$$

where

d_f = Design stillwater flood depth as defined in Section 5.3.3,

H_{design} = Design wave height as defined in Section 5.3.7,

L = Wavelength in ft (m) as defined in Section 5.3.7, and

S_m Need not be taken greater than S_m as calculated in Section 5.3.8.1.2.

5.3.8.1.2 Scour Due to Breaking Waves

The maximum scour depth, S_m, in ft (m) at walls for breaking waves shall be calculated by Equation (5.3-12):

$$S_m = H_{design} \quad (5.3\text{-}12)$$

where H_{design} = breaking wave height H_b in ft (m) as defined in Section 5.3.7.

5.3.8.2 Scour at Vertical Piles and Columns

The scour depth, S_m, at the base of a single vertical pile or column inundated at the time of flooding shall be calculated by Equation (5.3-13). Tightly spaced piles shall be considered to act as a wall per Section 5.3.8.1.

$$S_m = 2.0\, D \quad (5.3\text{-}13)$$

where D = pile or column diameter, in ft (m) for circular sections, or for a square pile or column, 1.4 times the width of the pile or column in ft (m).

Scour of fully embedded piles (i.e., no exposed length of pile above ground) shall only be considered if scour is determined to extend below the bottom of the overlying structure, including the pile cap(s). Scour shall be calculated in accordance with Section 5.3.8.1 for overlying walls and in accordance with Section 5.3.8.2 for overlying columns.

5.3.9 Debris

Risk Categories II, III, and IV structures shall be designed for debris impact and debris damming in accordance with this section where the Design Stillwater Flood Depth (d_f) is greater than 3 ft (0.91 m).

5.3.9.1 Debris Impact.

Structures within the Flood Hazard Area shall be designed for debris impact loads as determined by Section 5.4.5.1. Debris impact loads shall be considered in any direction and at heights as required per Section 5.4.5.2. Debris impact loads need not be considered on multiple structural elements simultaneously.

EXCEPTIONS:

1. Where a site-specific study provides flow directions, debris impacts need only be considered from the directions shown in the site-specific study +/- 22.5 degrees.

2. For riverine sites, debris strikes need only be considered from the upstream direction with a strike direction of +/- 22.5 degrees from the primary direction of flow.

3. Design for debris impact is not required for detached one- and two-family dwellings.

4. Design for debris impact is not required for Risk Category II structures outside of the Special Flood Hazard Area.

5.3.9.1.1 Debris Impact Objects

Buildings and other structures as required in Section 5.3.9 shall be designed for debris impact loads from floating debris. Debris objects considered shall be determined based on risk category, flow depth, and structure component type in accordance with Table 5.3-4. A site hazard assessment per Section 5.3.9.1.2 shall be used to determine the applicability of debris strikes as required. Any ship or barge exceeding 88,000 lb (391 kN), based on the Lightweight Tonnage (LWT) plus 30% of Deadweight Tonnage (DWT), shall be considered as Extraordinary Debris.

Table 5.3-4 Debris Type Applicability.

Debris Type	Applicable Risk Categories	Threshold Depth (ft)[1]	Impact on columns, piles, bearing walls and transfer beams	Impact on non-load bearing elements[2]
Passenger Vehicles	RC II/III/IV	3 ft (0.91 m)	Yes	Yes
Small Vessels	RC II/III/IV	3 ft (0.91 m)	Yes[3]	Yes[3]
Wood Poles	RC III/IV	3 ft (0.91 m)	Yes	Yes
Shipping Containers	RC III/IV	3 ft (0.91 m)	Yes[3]	n/a
Ships/barges	RC III/IV	6 ft (1.8 m)	Yes[3]	n/a
Extraordinary Debris[3]	RC IV	12 ft (3.7 m)	Yes[3 4]	n/a

[1]Threshold depth is the minimum Design Stillwater Flood Depth required for a debris object to be considered for design.

[2]Elements that are part of a dry floodproofing system, including temporary flood barriers, as defined by ASCE 24.

[3] As required by Section 5.3.9.1.2.

[4]See Section 5.3.9.1.3 for applicable impacted elements.

5.3.9.1.2 Site Hazard Assessment for Localized Marine Debris, Shipping Containers, Ships, Small Vessels, and Barges

Nearby container yards, ports/harbors, marinas, or other sources shall be evaluated as potential debris origins for shipping containers, ships, small vessels, and barges according to the following procedure.

Debris travel from their source location shall be based on a two-step process. Debris shall be assumed to travel over water, beach or open land and shall travel a minimum of a 10,000 ft (3.03 km) radius from the source or until rougher land surface exists. Then, from any point within the initial debris spread area the debris can travel an additional distance into a developed environment in accordance with Table 5.3-5.

Table 5.3-5. Debris Travel in Urban Environment.

Debris Type	Travel distance in moderate density environment [*]	Travel distance in heavy density environment [1]
Small vessels	2,000 ft (604 m)	1,000 ft (304 m)
Shipping Containers	2,000 ft (604 m)	1,000 ft (304 m)
Ships/barges	1,000 ft (302 m)	500 ft (152 m)
Extraordinary debris	1,000 ft (302 m)	500 ft (152 m)

[*]Heavy density environments are areas where the density of structures with a height of at least 75% of the design flood depth is greater than 30% of plan area within the Flood Hazard Area. All other areas shall be considered moderate density.

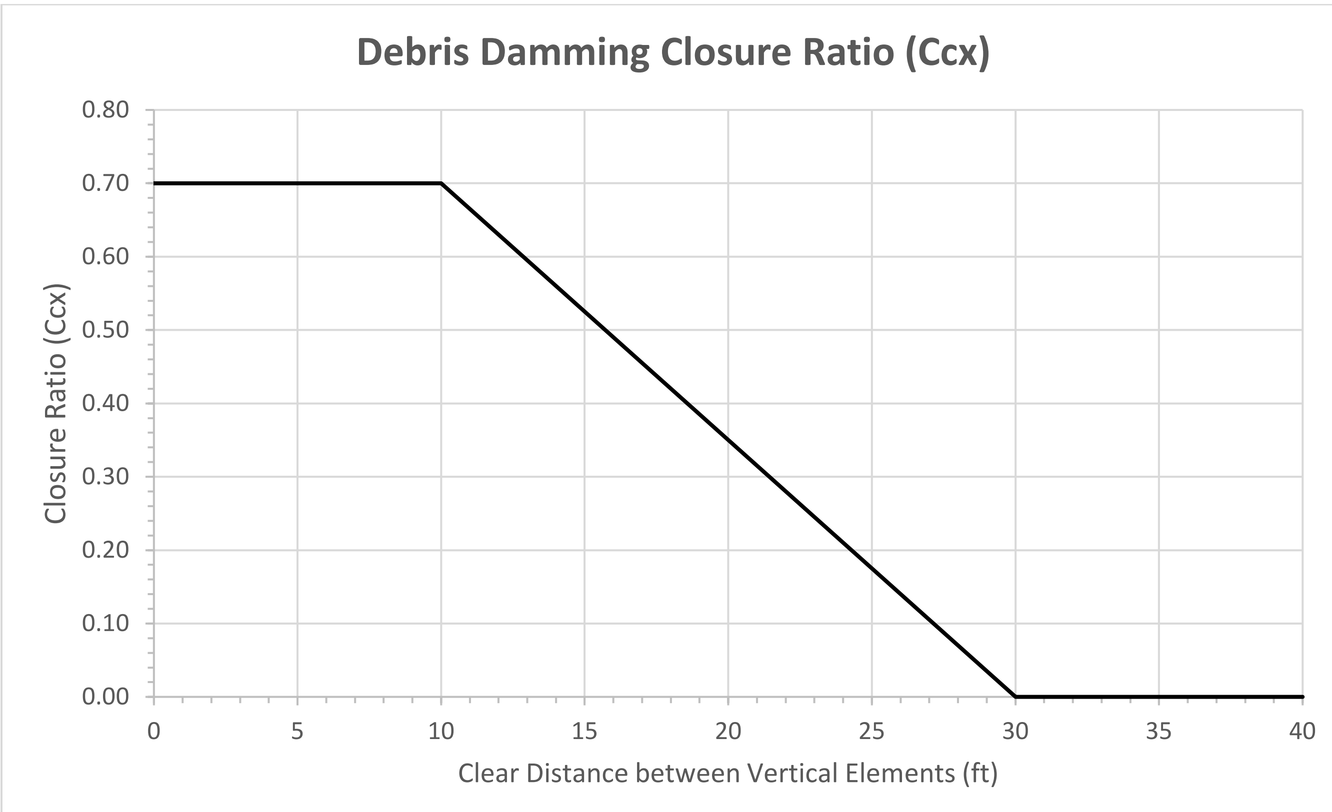
EXCEPTION: Debris impact loads from shipping containers, ships, and barges need not be considered where the flood depth at the structure is less than the draft of the debris object plus 2.0 ft (0.61 m), or where the path of the debris object is blocked by a structure or topographic feature that results in inadequate draft.

5.3.9.1.3 Extraordinary Debris Impact Loading

As required by Section 5.3.9.1.1 all perimeter columns, bearing walls and transfer beams shall be designed for the largest ship or barge exceeding 88,000 lb (391 kN) as required by Section 5.4.5.3.

5.3.9.2 Debris Damming

The effects of debris damming shall be applied to structures that allow the free passage of flood waters through the building footprint, including structures with breakaway walls, as required by this section. A minimum closure ratio (C_{cx}) shall be determined in accordance with Figure 5.3-1 based on the clear spacing between structural columns, piles and/or walls perpendicular to the flow direction. The closure ratio shall be used to determine the drag load on vertical structural elements for hydrodynamic loading as required in Section 5.4.3.



Note:: To convert to SI units multiply feet by 0.305 to get clear distance in meters.

Figure 5.3-1. Debris Damming Closure Ratio (C_{cx}).

5.3.10 Loads on Breakaway Walls.

Where required by ASCE/SEI 24 to break away, walls and partitions, including their connections to the structure, shall be designed in accordance with this section. The wall shall be designed to resist the following loads acting perpendicular to the plane of the wall:

1. The wind load specified in this standard,
2. The seismic load specified in Chapter 12
3. The lateral earth pressure specified in Chapter 3, and
4. 16 lb/ft^2 (0.76 kN/m^2).

If the largest of the loads above is less than the 100-year flood load, the wall shall be designed to fail during the 100-year flood condition.

The structure and its foundation shall be designed against collapse, permanent lateral displacement, and other structural damage due to the expected failure forces as walls break away.

5.3.11 Site-Specific Studies

If site-specific hydrologic and hydraulic studies are used to estimate effects from floods, they shall be conducted in accordance with accepted engineering practice. Where a site-specific study is performed, a report shall be prepared and submitted to the Authority Having Jurisdiction. The report shall provide the proposed value of each parameter shown in Table 5.3-6 and the basis for these values. Maximum allowable reductions in velocity, wave height, and wave period at the site are given in Table 5.3-6. The reduction in the velocity and the design wave height and period shall be based on the velocity, wave height, and period computed from Section 5.3.6 and Section 5.3.7, respectively.

Table 5.3-6. Maximum Allowable Reductions for Site-Specific Studies.

Hazard	Allowable Reduction with Peer Review	Allowable Reduction without Peer Review
Velocity, V	30%	20%
Wave height, H	30%	20%
Wave period, T	30%	20%

5.3.12 Performance-Based Design.

Flood design of buildings and other structures using performance-based procedures shall be permitted subject to the approval of the Authority Having Jurisdiction. The performance-based flood design procedures used shall, at a minimum, conform to Section 1.3.1.3.

5.4 LOADS DURING FLOODING

5.4.1 Load Basis

In the Flood Hazard Area~~s~~, the flood loads used for structural design shall be based on the Design Flood.

5.4.2 Hydrostatic Loads

Hydrostatic loads caused by a depth of water to the design stillwater flood elevation shall be applied over all surfaces contacted, both above and below ground level.

Hydrostatic forces shall be calculated in accordance with Sections 5.4.2.1 and 5.4.2.2. The hydrostatic pressures shall be calculated utilizing basic fluid mechanics by applying pressures perpendicular to wetted surfaces proportional to the depth of water such that

$$p_h = \gamma_w z \qquad (5.4\text{-}1)$$

where

p_h = Hydrostatic pressure at a given depth z, in lb/ft^2 (kN/m^2),

γ_w = Specific weight of water, taken as 62.4 lb/ft^3 (9.81 kN/m^3) for freshwater and 64 lb/ft^3 (10.03 kN/m^3) for saltwater, and

z = Depth below design stillwater flood elevation, in ft (m).

Reduced hydrostatic uplift and lateral loads on surfaces of enclosed spaces below the design stillwater flood elevation shall apply only if provision is made for entry and exit of floodwater.

Hydrostatic loads below grade shall be calculated assuming the soils are fully saturated and in accordance with Sections 5.4.2.1 and 5.4.2.2 unless the degree of soil saturation and below grade porewater pressures during a flood event are determined in accordance with Section 5.4.2.3.

5.4.2.1 Vertical Hydrostatic Force

Structures or portions of a structure submerged below the design stillwater flood elevation and with enclosed air in spaces where walls are not designed to break away or allow for entry and exit of floodwaters shall be designed for buoyancy. Buried portions of structures shall be designed assuming the soils beneath the structure are fully saturated unless otherwise determined per Section 5.4.2.3.

The vertical uplift force caused by buoyancy for determination of structure uplift shall be applied at the centroid of the submerged volume of the structure, and shall be calculated using Equation (5.4-2):

$$F_B = \gamma_w \mathrm{V_w} \qquad (5.4\text{-}2)$$

where

F_B = Uplift force caused by buoyancy, in lb (kN),

γ_w = Specific weight of water, taken as 62.4 lb/ft^3 (9.81 kN/m^3) for freshwater and 64 lb/ft^3 (10.03 kN/m^3) for saltwater, and

V_w = Volume of displaced water, in ft^3 (m^3).

5.4.2.2 Lateral Hydrostatic Force

The lateral force, F_h, caused by the hydrostatic pressure on one side of a vertical wall per unit width, lb/ft (kN/m) shall be calculated by Equation (5.4-3):

$$F_h = (1/2)\ \gamma_w\ d_f^2 \qquad (5.4\text{-}3)$$

5.4.2.3 Seepage

Numerical modeling for analysis of transient seepage is permitted to evaluate the porewater pressures on structures for conditions before, during, and after a flood event in lieu of using hydrostatic pressures as described in Sections 5.4.2.1 and 5.4.2.2. Permeability of soils and impermeable cutoffs (e.g., concrete slabs, below-grade walls) shall be included in the seepage analysis. The seepage analysis shall be based on a geotechnical investigation report that includes test data for soil permeability.

5.4.3 Hydrodynamic Loads

5.4.3.1 Drag Force on Components

The hydrodynamic drag force, exerted by moving water on structural components immersed in the flow and buildings immersed in the flow, shall be determined by Equation (5.4-4):

$$F_{drag} = (1/2)\ \rho\ C_d\ V^2\ h\ (b + C_{cx}\ s) \qquad (5.4\text{-}4)$$

where

ρ = Mass density of water, in lb s^2/ft^4, taken as 1.94 lb s^2/ft^4 (1000 kg/m^3) for fresh water and 1.99 lb s^2/ft^4 (1027 kg/m^3) for seawater,

C_d = Drag coefficient for submerged objects subjected to currents, defined in Table 5.4-1,

V = Design flood velocity, in ft/s (m/s),

h = Submerged height of column or wall above its foundation or structural floor level, in ft (m),

b = Width of column or wall, perpendicular to the direction of flow considered, in ft (m),

C_{cx} = Debris damming closure ratio as determined in Section 5.3.9.2, and

s = Average clear spacing of column or wall to the adjacent column or wall, in ft (m). For corner columns, half the clear distance shall be used.

For geometries not provided in Table 5.4-1 and for structures with openings, the dynamic effects of moving water shall be determined by a detailed analysis utilizing concepts of fluid mechanics.

Table 5.4-1. Drag Coefficients for Structural Components.

Structural Element Section	Drag Coefficient, C_d
Round column or equilateral polygon with six sides or more	1.2
Rectangular column of at least 2:1 aspect ratio with longer face oriented parallel to flow	1.6
Free-standing wall submerged parallel to flow	1.6
Square or rectangular column with longer face oriented perpendicular to flow	2.0
Triangular column pointing away from flow	2.0
Wall or flat plate, normal to flow	2.0
Diamond-shape column, pointed into the flow (based on face width, not projected width)	2.5
Rectangular beam, normal to flow	2.0
I, L, and channel shapes	2.0
Structural components with debris damming per Section 5.3.9.2	2.0

5.4.3.2 Drag Force on Lateral Force Resisting System.

The building lateral force resisting system shall be designed to resist the overall drag forces on the structure. For enclosed buildings the lateral force shall be determined based on Equation 5.4-5:

$$F_{drag} = (1/2)\, \rho\, C_d\, V^2\, B\, d_f \qquad (5.4\text{-}5)$$

ρ = Mass density of water, in lb s^2/ft^4, taken as 1.94 lb s^2/ft^4 (1000 kg/m^3) for fresh water and 1.99 lb s^2/ft^4 (1027 kg/m^3) for seawater,

C_d = Drag coefficient for per Table 5.4-2 for Rectilinear Buildings and Structures,

V = Design flood velocity, in ft/s (m/s),

B = Overall width of building perpendicular to the flow direction, in ft (m), and

d_f = Design flood depth per Section 5.3.3, in ft (m).

When the design flood depth d_f exceeds the height of the structure, then d_f in Equation 5.4-5 is replaced by the height of the structure.

For open buildings or buildings with breakaway walls as defined by Section 5.3.10, the building lateral force resisting system shall be designed to resist the summation of the drag loads of each exposed vertical and horizontal element as required by Section 5.4.3.1 using Equation 5.4-4. The effects of debris damming shall be considered on the side of the structure exposed to the flow direction, considering damming on any two adjacent bays or a minimum of a 50 ft (15.2 m) width, whichever produces the largest base shear.

Table 5.4-2. Drag Coefficients for Rectilinear Buildings and Structures.

Ratio of structure width to design stillwater flood depth* B/d_f	Drag Coefficient, C_d
≤ 12	1.25
≥ 120	2.0

*Linear interpolation shall be used for intermediate values of b/d_f. Where building setbacks occur, drag coefficients shall be determined for each portion of constant width.

5.4.4 Wave Loads.

Wave loads shall be determined by one of the following two methods: (1) by using the analytical procedures outlined in this section, or (2) by more advanced analytical procedures, numerical modeling procedures, or physical modeling procedures. If more advanced procedures are used to determine wave loads, any reduction in wave loads from those in this section shall not exceed 20 percent without conducting a peer review. In no case shall a reduction of more than 30 percent be permitted.

Waves shall be evaluated per Section 5.3.7.

For the calculation of wave loads per Section 5.4.4, a structural element with the ratio of the design stillwater flood depth to the lateral dimension facing the wave that is greater than or equal to three shall be considered a pile or column, and wave loads on the member shall be calculated per Section 5.4.4.1. A structural element with a ratio of the design stillwater flood depth to the lateral dimension facing the wave that is less than three shall be considered to act as a wall, and wave loads on the element shall be calculated per Section 5.4.4.2. Tightly spaced piles or columns where the clear distance between piles or columns is less than one half of the lateral dimension of an individual pile or column facing the wave shall be considered to act as a wall.

5.4.4.1 Wave Loads on Vertical Piles and Columns

Lateral wave loads on vertical piles or columns shall be calculated using the procedures described in Section~~s~~ 5.4.4.1.1 for nonbreaking waves and Section 5.4.4.1.2 for breaking waves. Broken wave loads shall be treated similar to nonbreaking wave loads.

5.4.4.1.1 Nonbreaking Wave Loads on Vertical Piles and Columns

Equations for nonbreaking wave loads on vertical piles and columns shall be considered applicable when the nondimensional parameter $W = C_M D/(C_D H_{design})$ is less than or equal to 1.0, where C_M is an inertia coefficient, taken as 2.0 for round piles or columns and taken as 2.5 for square piles or columns. When W is greater than 1.0, then the wave load shall be calculated using the requirements of Section 5.4.4.2.1.

When W *is* less than or equal to 1.0, the maximum net lateral force resulting from a nonbreaking wave F_m in lbs (kN) acting on a vertical pile or column shall be calculated by Equation (5.4-6) and applied at the design stillwater flood elevation:

$$F_m = \phi_m C_D \gamma_w H_{design}^2 D \tag{5.4-6}$$

where

ϕ_m = Force coefficient, taken as 0.5 for round or square piles and round or square columns,

C_D = Wave drag coefficient, shall be taken as 0.7 for round piles or columns and shall be taken as 2.25 for square or rectangular piles or columns,

H_{design} = Design wave height in ft (m) as defined in Section 5.3.7, and

D = Pile or column diameter, in ft (m) for circular sections, or the largest projected width of the pile or column in ft (length of plan diagonal) (m) for a square or rectangular pile or column.

5.4.4.1.2 Breaking Wave Loads on Vertical Piles and Columns

The maximum net lateral force F_{bw} resulting from a breaking wave acting on a vertical pile or column shall be assumed to act at the design stillwater flood elevation and shall be calculated by Equation (5.4-7):

$$F_{bw} = \phi_m C_{bw} \gamma_w H_{design}^2 D \qquad (5.4\text{-}7)$$

where

ϕ_m = Force coefficient, taken as 0.5 for round or square piles and round or square columns,

C_{bw} = Coefficient for breaking waves, shall be taken as 1.75 for round piles or columns and shall be taken as 2.25 for square or rectangular piles or columns. H_{design} = design wave height in ft (m) as defined in Section 5.3.7, and

D = Pile or column diameter, in ft (m) for circular sections, or the largest projected width of the pile or column in ft (length of plan diagonal) (m) for a square or rectangular pile or column.

5.4.4.2 Lateral Wave Loads on Walls

Lateral nonbreaking and broken wave loads on non-elevated vertical walls shall be calculated using the procedures described in Section 5.4.4.2.1. Lateral breaking wave loads on non-elevated vertical walls shall be calculated using the procedures described in Section 5.4.4.2.2. Lateral wave loads shall be modified for nonvertical walls and obliquely incident waves as specified in Section 5.4.4.2.3 and Section 5.4.4.2.4, respectively. Wave loads on elevated walls shall be calculated using the procedures described in Section 5.4.4.2.5.

As required in Section 5.5, the wave-induced hydrodynamic pressure distribution and forces in this section shall be combined with hydrostatic loads, in accordance with Section 5.4.2, and hydrodynamic loads, in accordance with Section 5.4.3. Hydrostatic or hydrodynamic loads shall be considered for conditions in which the design stillwater flood depth is not equal on both sides of the wall and/or a current affects the structure in accordance with the combination of flood load cases per Section 5.5.

5.4.4.2.1 Lateral Nonbreaking Wave Loads on Non-elevated Vertical Walls

The pressure distribution due to a nonbreaking wave on a vertical wall is shown in Figure 5.4-1 and shall be calculated using Equations (5.4-8) to (5.4-11):

$$\eta^* = 1.5H_{design} \qquad (5.4\text{-}8)$$

$$p_1 = [0.6 + 0.5\left(\frac{4\pi d_f/L}{\sinh(4\pi d_f/L)}\right)^2]\gamma_w H_{design} \qquad (5.4\text{-}9)$$

$$p_2 = \begin{cases} \left(1 - \frac{h_c}{\eta^*}\right) p_1 & \text{for } \eta^* > h_c \\ 0 & \text{for } \eta^* \leq h_c \end{cases} \tag{5.4-10}$$

$$p_3 = \left(\frac{1}{\cosh\,(2\pi d_f/L)}\right) p_1 \tag{5.4-11}$$

where

η^*= Height in ft (m), measured above the design stillwater flood elevation, below which the wave pressure is assumed to act, or the minimum height at which the wave pressure equals zero,

H_{design} = Design wave height as defined in Section 5.3.7, in ft (m),

p_1 = Pressure at the design stillwater flood elevation, in lb/ft^2 (kN/m^2),

p_2 = Pressure at the top of the vertical wall or structure, in lb/ft^2 (kN/m^2),

p_3 = Pressure at the eroded ground elevation, in lb/ft^2 (kN/m^2),

d_f = Design stillwater flood depth as defined in Section 5.2, in ft (m),

h_c = Height to the top of the vertical wall or structure above the design stillwater flood elevation, in ft (m), and

L = Wavelength as specified in Section 5.3.7, in ft (m).

The parameters $\left(\frac{4\pi df/L}{\sinh\,(4\pi df/L)}\right)^2$ and $\left(\frac{1}{\cosh\,(2\pi d_f/L)}\right)$ range between 0 and 1 and may conservatively be taken as equal to 1.0 in Equations (5.4-9) and (5.4-11).

The lateral wave-induced force per unit length on the vertical wall or structure in lb/ft (kN/m) shall be calculated using Equation (5.4-12):

$$F_t = \begin{cases} \left(\frac{1}{2}(p_1 + p_2)h_c + \frac{1}{2}(p_1 + p_3)d_f\right) & \text{for } \eta^* > h_c \\ \left(\frac{1}{2}p_1(\eta^*) + \frac{1}{2}(p_1 + p_3)d_f\right) & \text{for } \eta^* \leq h_c \end{cases} \tag{5.4-12}$$

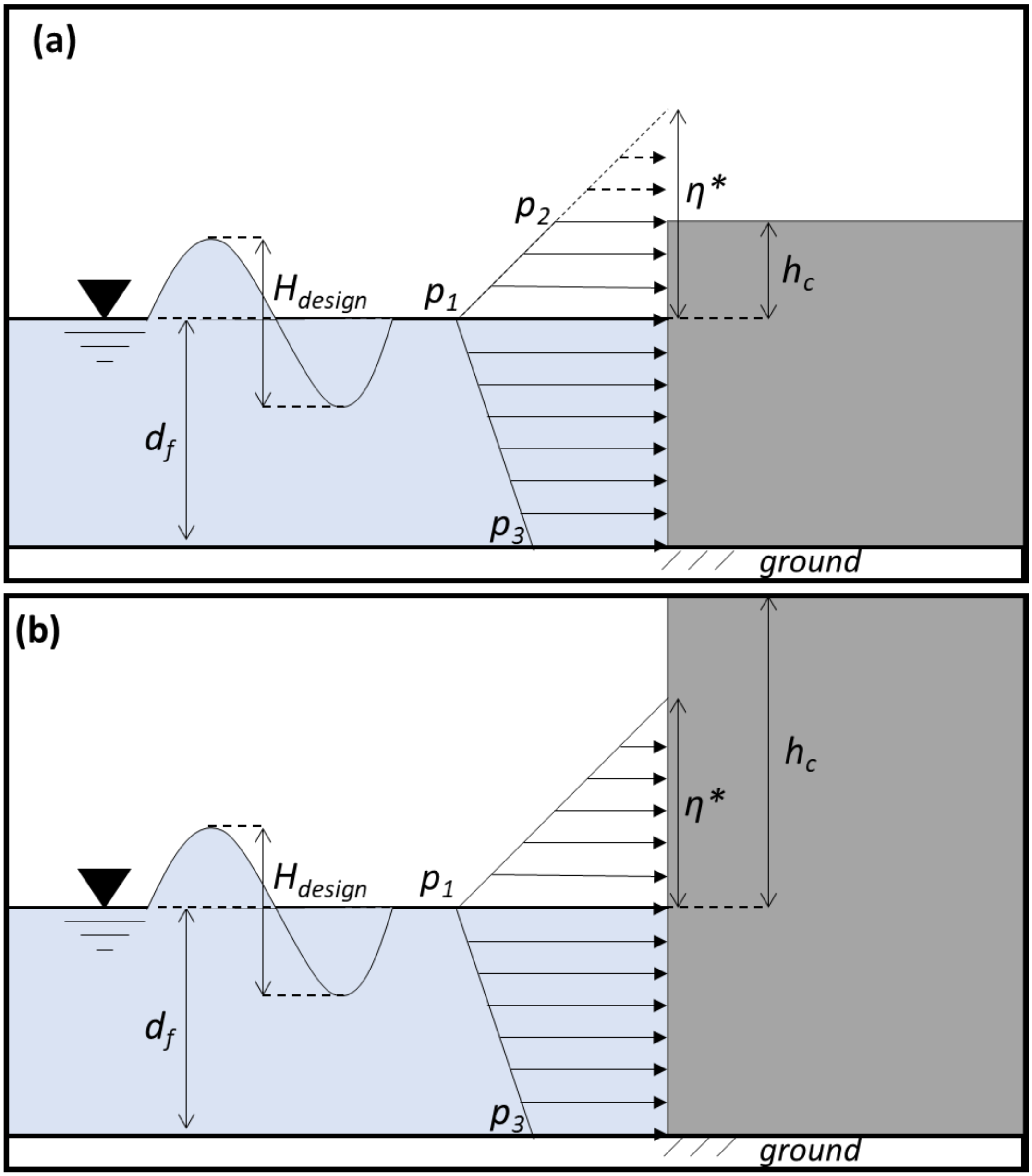


Figure 5.4-1. Normally incident wave pressures against a non-elevated vertical wall: (a) $\eta^* > h_c$; (b) $\eta^* \leq h_c$.

5.4.4.2.2 Lateral Breaking Wave Loads on Non-elevated Vertical Walls

The lateral breaking wave force per unit length on a vertical wall, F_{BRK} in lb/ft (kN/m) shall be calculated by Equation (5.4-13):

$$F_{BRK} = \begin{cases} \left(\frac{1}{2}(p_{1B} + p_2)h_c + \frac{1}{2}(p_{1B} + p_3)d_f\right) & \text{for } \eta^* > h_c \\ \left(\frac{1}{2}p_{1B}(\eta^*) + \frac{1}{2}(p_{1B} + p_3)d_f\right) & \text{for } \eta^* \leq h_c \end{cases} \qquad (5.4\text{-}13)$$

where

p_{1B} = Breaking wave pressure acting at the design stillwater flood elevation in lb/ft^2 (kN/m^2) calculated by Equation (5.4-14):

$$p_{1B} = \left[0.6 + 0.5\left(\frac{4\pi d_f/L}{sinh\,(\pi 4 d_f/L)}\right)^2 + \alpha^*\right]\gamma_w H_{design} \qquad (5.4\text{-}14)$$

where

α^*= Impulsive wave pressure coefficient, which shall be taken as 0.8, and

H_{design} = Design wave height in ft (m) as defined in Section 5.3.6.

In Equation (5.4-13), p_2, p_3, and η^* are calculated as prescribed in Section 5.4.4.2.1, and the parameter $\left(\frac{4\pi df/L}{sinh\,(4\pi df/L)}\right)^2$ may be conservatively taken as 1.

5.4.4.2.3 Lateral Breaking Wave Loads on Nonvertical Walls

Lateral breaking wave forces shall be modified in instances where the walls or surfaces upon which the breaking waves act are nonvertical. The horizontal component of the breaking wave force per unit length on a nonvertical wall, F_{BNV}, in lb/ft (kN/m) shall be calculated by Equation (5.4-15):

$$F_{BNV} = F_{BRK}\,[sin(\alpha_V)]^2 \qquad (5.4\text{-}15)$$

where

F_{BNV} = Horizontal component of breaking wave force per unit length on a nonvertical wall, in lb/ft (kN/m),

F_{BRK} = Breaking wave force per unit length on a vertical wall, in lb/ft (kN/m), and

α_V = Vertical angle between nonvertical surface and the horizontal.

5.4.4.2.4 Lateral Breaking Wave Loads from Obliquely Incident Waves

Lateral breaking wave forces shall be modified in instances where waves are obliquely incident. Breaking wave forces from obliquely incident waves shall be calculated by Equation (5.4-16):

$$F_{BOI} = F_{BRK}\,[sin(\alpha_H)]^2 \qquad (5.4\text{-}16)$$

where

F_{BOI} = Horizontal component of obliquely incident breaking wave force in lb/ft (kN/m),

F_{BRK} = Breaking wave force per unit length on a vertical wall, in lb/ft (kN/m), and

α_H = Horizontal angle between the direction of wave approach and the vertical surface.

5.4.4.2.5 Lateral Wave Loads on Elevated Walls

The procedures for determining lateral wave loads on elevated vertical walls due to nonbreaking or breaking waves shall be calculated by modifying the procedures for non-elevated vertical walls outlined in Sections 5.4.4.2.1 and 5.4.4.2.2, respectively, by considering only pressures that act over the vertical surface of the wall. Figure 5.4-2 shows a case when the lowest portion of the wall is above the design stillwater flood elevation (positive air gap) and a case when the design stillwater flood elevation is above the lowest portion of the wall (negative air gap).

The pressure distribution associated with nonbreaking waves interacting with an elevated wall shall be calculated in accordance with Section 5.4.4.2.1, with pressure at the base of the structure p_4 determined by linearly interpolating between p_1 and $p=0$ at η^* (positive air gap) or p_1 and p_3 (negative air gap), using Equation (5.4-17):

$$p_4 = \begin{cases} \left(1 - \frac{a}{\eta^*}\right) p_1, & \text{for } a \geq 0 \\ \left(1 - \frac{|a|}{d_f}\right)(p_1 - p_3) + p_3, & \text{for } a < 0 \end{cases} \tag{5.4-17}$$

where a = Air gap between lowest supporting horizontal structural member of lowest above grade floor and the design stillwater flood elevation, in ft (m).

The total lateral wave-induced force per unit length of the elevated wall in lb/ft (kN/m) shall be calculated by integrating the pressure distribution over the vertical domain of the wall. For conditions in which no wave overtopping occurs, the total lateral force per unit length is given by Equation (5.4-18):

$$F_t = \begin{cases} \left(\frac{1}{2} p_4 (\eta^* - a)\right), & \text{for a} \geq 0 \\ \left(\frac{1}{2} p_1 \eta^* + \frac{1}{2}(p_1 + p_4)(|a|)\right), & \text{for } a < 0 \end{cases} \tag{5.4-18}$$

Lateral breaking wave loads on elevated walls shall be calculated in accordance with the above equations, applying the modified p_{1B} instead of p_1 in Equations (5.4-17) and (5.4-18). Modified pressure p_{1B} shall be calculated as described in Section 5.4.4.2.2. Wave loads on elevated nonvertical walls or due to obliquely incident waves shall be modified as described in Sections 5.4.4.2.3 and 5.4.4.2.4, respectively.

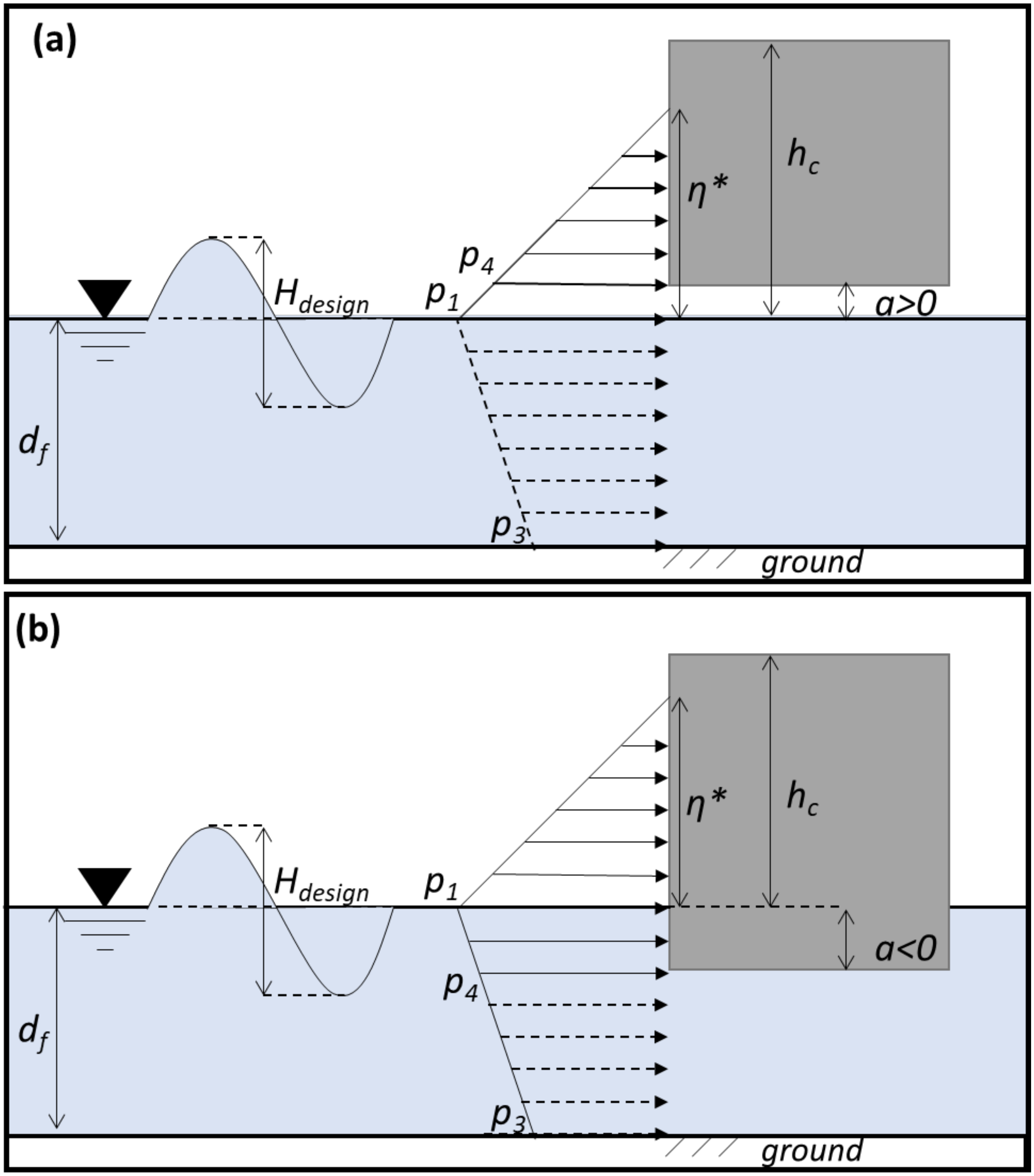


Figure 5.4-2. Normally incident wave pressures on an elevated wall: (a) $a \geq 0$; (b) $a < 0$.

5.4.4.3 Wave Uplift Forces on Elevated Structures and Non-Elevated Structures with Overhangs

Wave uplift forces on elevated structures shall be considered on portions of elevated structures located less than $0.7H_{design}$ above the design stillwater flood elevation.

Wave uplift forces on horizontal overhangs shall be considered when overhangs are positioned within a height of η^* above the design stillwater flood elevation and a solid wall below directs water up against the underside of the overhang.

5.4.5 Debris Impact Loads

Debris impact loads shall be applied as required by Sections 5.3.9.1 and 5.4.5.2 and shall be determined in accordance with this section.

5.4.5.1 Debris Impact Load Determination

Debris impact forces shall be determined by one of the approaches in Sections 5.4.5.1.1 through 5.4.5.1.3. It is permitted to use any of these methods based on their applicability per debris type.

5.4.5.1.1 Simplified Debris Impact Load for Passenger Vehicles or Small Vessels

Debris impact forces for passenger vehicles or small vessels shall be permitted to be determined by applying a static lateral force given by Equation (5.4-19) as the load in lieu of the loads based on the other methods of this section:

$$F_{di} = C_o \ 51{,}000 \ (\text{lb}) \qquad (5.4\text{-}19)$$

$$F_{di} = C_o \ 227 \ (\text{kN}) \qquad (5.4\text{-}19.\ \text{SI})$$

where C_o = debris orientation coefficient, taken as 0.80.

5.4.5.1.2 Elastic Debris Impact Loads

Debris impact forces, F_{di} in lb (kN) are permitted to be calculated using the elastic debris impact method per Equation (5.4-20):

$$F_{di} = C_o \ V \ C_R C_s \ (k_e \ m_{debris})^{0.5} \quad (5.4\text{-}20)$$

where

C_o = Debris orientation coefficient, taken as 0.80,

V = Design flood velocity as defined in Section 5.3.6, in ft/s (m/s),

C_R = Debris depth coefficient taken as 1.0 for design flood depths greater than 5 ft (1.52 m) and taken as 0.0 for design flood depths less than 1 ft (0.3 m). Linear interpolation is permitted between design flood depths of 1 ft (0.3 m) and 5 ft (1.52 m), and

C_s = Debris velocity stagnation coefficient per Table 5.4-3. Applicable for nonload-bearing elements on the exterior of buildings along the front face of a building wider than 30 ft (9.14 m). Walls must extend from grade to above the design stillwater flood elevation and be designed for the flood loads of this chapter. For load bearing elements Cs shall be taken as 1.0.

Table 5.4-3. Debris Velocity Stagnation Coefficient, C_s.

Location on building	Debris velocity stagnation coefficient (C_s)
Within a distance of the greater of 0.2*B* or 10 ft (3.05 m) at edges of building	1.00
Middle 0.6*B* of building	0.50

Notes:

B = Overall width of building perpendicular to flow direction, in ft (m).

k_e = effective stiffness of the impacting debris or the effective lateral stiffness of the impacted structural element(s) deformed by the impact, in lb/ft (kN/m) determined in accordance with Section 5.4.5.2. It is permissible to use the combined elastic stiffness of the debris and the impacted element in series.

m_{debris} = mass of the debris (W_{debris}/g) in lb s^2/ft (kg) determined in accordance with Section 5.4.5.2.

5.4.5.1.3 Alternative Methods of Debris Impact Analysis

Structures or elements of structures are permitted to be modeled as an equivalent single degree of freedom mass-spring system with a nonlinear stiffness that considers the ductility of the impacted structure or element for the dynamic analysis. Alternatively, the structural response shall also be permitted to be calculated based on a work-energy method with nonlinear stiffness that incorporates the ductility of the impacted structure. The velocity used for this analysis shall be $VC_RC_s\ C_o$ as defined by Section 5.4.5.1.2. Debris impact loads as defined in Section 5.3.9.1 and Section 5.4.5.2 shall be applied to the structure to produce the most critical flexural and shear demands.

5.4.5.2 Debris Types and Properties

Debris impact forces specified in Section 5.3.9.1.1 shall have the following minimum application requirements:

1. Wood poles/logs strike in their longitudinal direction centered at any elevation from 3 ft (0.91 m) above grade up to the design stillwater flood depth with an impact area of 1.5 ft (0.46 m) by 1.5 ft (0.46 m).
2. Passenger vehicles strike the structure centered at any elevation from 3 ft (0.9 -m) above grade to 1 ft (0.3 m) below the design stillwater flood depth with an impact area of 5 ft (1.5 m) wide by 2 ft (0.61 m) high.
3. Small vessels strike the structure centered at any elevation from 3 ft (0.91 m) above grade to 3 ft (0.91 m) above the design stillwater flood depth with an impact area of 4 ft (1.2 m) wide by 2 ft (0.61 m) high.
4. The bottom corner rails of shipping containers strike the structure at an elevation from 3 ft above grade to the design stillwater flood depth with an impact area of 1 ft (0.3 m) by 1 ft (0.3 m).

The impact force from ships and barges of less than 88,000 lb (39,916 kg) lightweight tonnage (LWT) shall be determined in accordance Section 5.4.5.1.2 or 5.4.5.1.3. The debris strike shall be applied in the longitudinal direction of the debris. The minimum mass considered shall be the mass of the largest expected ship or barge required by Section 5.3.9.1.2. The stiffness of the ship or barge shall be estimated based on the length, width, and construction type. The impact force shall be permitted to be limited to the crushing strength of the ship or barge.

The mass and stiffness to be used for debris impact in Sections 5.4.5.1.2 and 5.4.5.1.3 shall not be less than the minimum values provided in Table 5.4-4.

Table 5.4-4. Minimum Debris Properties.

Debris Type	Minimum debris weight (W_{debris})	Minimum elastic debris stiffness (k_e)
Wood Log/Pole	1,000 lb (4.448 kN)	4,200,000 lb/ft (61,300 kN/m)
Passenger Vehicle	2,400 lb (12.455 kN)	72,000 lb/ft (1,051 kN/m)
Small Vessels	2,500 lb (11.121 kN)	360,000 lb/ft (5,254 kN/m)
20 ft Shipping Container	5,000 lb (22.241 kN)	2,940,000 lb/ft (42,900 kN/m)

40 ft Shipping Container	8,400 lb (37.365 kN)	2,040,000 lb/ft (29,800 kN/m)
Ships/Barges	Established based on local conditions	

Where debris impacts from shipping containers, ships or barges exceed the capacity of the structural element, it shall be permitted to accommodate the impact through the provisions in Section 5.4.5.4. The provisions of Section 5.4.5.4 may be used for all debris impact types on individual piles.

5.4.5.3 Extraordinary Debris Impact

Extraordinary debris impact loading shall be based on the stiffness and weight in accordance with Section 5.3.9.1.1. The debris impact force shall be applied for the worst-case location on the impacted element from a depth of the eroded grade up to the design stillwater flood depth plus the maximum freeboard of the vessel. The load from the extraordinary debris shall be calculated from Equation (5.4-20) or alternative analysis of Section 5.4.5.1.3.

EXCEPTION: Either as the primary approach, or where the impact loads from extraordinary debris exceed the capacity of any structural element subject to impact, it shall be permitted to accommodate the impact through the progressive collapse provisions of Section 5.4.5.4.

5.4.5.4 Debris Impact Load Redistribution

Where permitted by Section 5.4.5.2 or 5.4.5.3, alternate load path progressive collapse provisions in accordance with the recognized literature shall be applied to perimeter columns, piles, and beams from ground level to the Design Stillwater Flood Depth plus the height of the deck of the vessel from the waterline where applicable.

5.5 FLOOD LOAD CASES

The flood load (F_a) used in the Chapter 2 load combinations shall include the following flood load cases in the applicable directions:

For coastal flooding:

1. Combination of hydrostatic loads including buoyancy (5.4.2), hydrodynamic loads (5.4.3) and debris impact loads (5.4.5)
2. Combination of hydrostatic loads including buoyancy (5.4.2), hydrodynamic loads (5.4.3) and wave loads (5.4.4)

For riverine flooding:

1. Combination of hydrostatic loads including buoyancy (5.4.2), hydrodynamic loads (5.4.3) and debris impact loads (5.4.5)

5.5.1 Stability Against Uplift.

Structures shall be designed to resist flotation due to buoyancy forces as defined in Section 5.4.2.1. Uplift resistance shall be provided by satisfying Equation (5.5-1) with load factors as shown. This stability load combination is in addition to those in Chapter 2.

$$0.9D_{SW} + R_B + F_B + 0.6W_{uplift} \geq 0 \qquad (5.5\text{-}1)$$

where

D_{SW} = Self-weight of the structure or portion of structure being evaluated inclusive of permanent fixed elements and equipment, in lb (kN),

R_B = Allowable uplift resisting capacity of structural foundation elements and/or other conditions resisting uplift, in lb (kN),

F_B = Uplift force caused by buoyancy in lb (kN), always taken as less than zero, and

W_{uplift} = Maximum total vertical uplift wind load on the structure as defined in this standard, in lb (kN). Wind load shall not be used to counteract buoyancy and is always taken less than zero.

5.5.2 Stability Against Sliding

Sliding resistance shall be provided by satisfying Equation (5.5-2) with load factors as shown. This stability load combination is in addition to those in Chapter 2:

$$\mu(0.9D_{SW} + F_B + 0.6W_{uplift}) + H_p + R_p - H_a - F_{lateral} - 0.6W_{lateral} \geq 0 \qquad (5.5\text{-}2)$$

where

μ = Coefficient of sliding friction at slip plane being considered between structure on shallow foundations and subgrade,

D_{SW} = Self-weight of the structure or portion of structure being evaluated inclusive of permanent fixed elements and equipment, in lb (kN),

F_B = Uplift force caused by buoyancy in lb (kN) always taken as less than zero,

W_{uplift} = Maximum total vertical uplift wind load on the structure as defined in this standard, in lb (kN). Wind load shall not be used to counteract buoyancy and is always taken less than zero,

H_p = Resultant force from passive lateral earth pressures, in lb (kN),

R_p = Allowable lateral resisting capacity of deep foundations, external structural foundation elements and/or other conditions resisting sliding, in lb (kN),

H_a = Resultant force from active lateral earth pressures, in lb (kN),

$W_{lateral}$ = Maximum total lateral wind load on the structure as defined in this standard, in lb (kN), and

$F_{lateral}$ = Maximum lateral component of the flood load, Fa, as determined in Section 5.5, in lb (kN).

Global sliding shall be evaluated for whichever directions of wind load and flood loads result in the more severe loading case. The effects of scour on the depth of soil contributing to lateral earth pressures shall be included in this evaluation.

5.6 CONSENSUS STANDARDS AND OTHER AFFILIATED CRITERIA

This section lists the consensus standards and other affiliated criteria that shall be considered part of this standard to the extent referenced in this chapter.

ASCE/SEI 24 *Flood resistant design and construction,* ASCE, 2014.

Cited in: Section 5.3.9, 5.3.10

COMMENTARY

Chapter C1: General

C1.3.1.3 Performance-Based Procedures

….. For seismic design, the provisions of the ASCE 41 standard and of the Tall Buildings Imitative, Guidelines for Performance-Based Seismic Design of Tall Buildings (PEER 2010) were either calibrated by structural performance level or were demonstrated in comparison with prescriptive design methods to provide reliabilities equal or better than Table 1.3-2.

In previous editions of this standard (2022 and previous), structures within the 100-year Coastal A zones and V zones were subject to a load factor of 2.0 that was based on a beta value of 2.5 (Mehta et al. 1998) rather than 3.0. A reliability analysis by the Flood Load Subcommittee for ASCE 7-22 Supplement 2 confirmed that using the 100-year design flood produced a beta of less than 2.5, which is below the target reliabilities outlined in this standard. As a result, the basis of design was changed to a risk-based approach using mean recurrence intervals associated with different risk categories of structure to achieve the target reliability.

Chapter C2: Combinations of Loads

C2.3.2 Load Combinations Including Flood Load

In ASCE 7-22 Supplement 2, the flood load requirements of Chapter 5 are updated from a 100-year hazard basis, which was developed for the FEMA National Flood Insurance Program (NFIP), to a risk category (RC) targeted return period basis (100-year, 500-year, 750-year, and 1,000-year flood for Risk Categories I, II, III, and IV, respectively). At the time of the change, longer return period flood maps (specifically 750-year and 1,000-year maps) are not available for all communities. Therefore, a scale factor for the flood loading is provided in Section 5.3, which approximates the longer return period floods from the 100-year mapped flood. These scale factors were established based on data from simulated storms impacting the Gulf and Atlantic coasts based on USACE mapping projects (Refer to Section C5.3). This change in approach, along with revised loading equations, is a significant departure from previous versions of ASCE 7. With this change, the load factor of 2.0 previously used for coastal flood hazard areas, with significant uncertainty relative to the base flood, is no longer necessary. Based on the increased return period and the inherent conservatism in some of the equations of Chapter 5, a load factor of 1.0 is appropriate for LRFD design. Similar to other hazards, the ASD load factor is taken as 0.7 (1/1.5 rounded to one significant digit) of the LRFD load factor. The load factor of 1.0 r was justified with a reliability analysis. The reliability analysis incorporated the revised return period requirements and loading criteria in Chapter 5 using hazard curve data from the USACE studies. The analysis considered a column structure and a wall within a structure located at several elevations above the datum. The target reliabilities were taken from Chapter 1, consistent with other environmental hazards such as wind and seismic. The uncertainty in the flood load design is based on several variables, including design stillwater flood elevation, velocity, and wave height. The reliability analysis accounts for these variables.

While flooding can occur in storms that produce tornadoes the spatiotemporal correlation between tornadoes and flooding is much less than that between hurricane winds and flooding. Therefore, the load combinations from Section 2.3.2 maximizing flood loads do not include arbitrary point-in-time tornado loads.

The reliability analysis performed for ASCE 7-22 Supplement 2 did not incorporate wind loading or below grade soil pressures. Therefore, the load factors for these elements remain unchanged. The 1.6 load factor specified for H loads (below-grade soil) in Section 2.3.1 is applied for groundwater induced lateral and uplift pressures for structural element design. Section 2.3.7

provides an alternative method for loads from water in soil. It should be noted that flood water and groundwater are not necessarily connected. This relationship depends on the subsurface soil characteristics. Stability for global uplift does not use the 1.6 load factor and should be checked per Section 5.5.1.

C2.4.2 Load Combinations Including Flood Load

See Section C2.3.2. The multiplier on F_a aligns allowable stress design for flood load with strength design.

Chapter C5: Flood Loads

C5.1 GENERAL

This chapter presents information for the design of buildings and other structures in areas prone to flooding. Design professionals should be aware that there are important differences between flood characteristics, flood loads, and flood effects in riverine and coastal areas (e.g., the potential for wave effects is much greater in coastal areas, the depth and duration of flooding can be much greater in riverine areas, the direction of flow in riverine areas tends to be more predictable, and the nature and amount of flood-borne debris varies between riverine and coastal areas).

This chapter is applicable to the vertical and lateral force resisting systems of buildings and other structures, and to the elements of a dry flood-proofed building such as cladding, opening barriers, and other associated components forming the flood-resistant perimeter. In addition, the phrase "other structures" is meant to cover building-like structures designed to resist flood-related loads. This standard does not cover levees, dikes, piers, wharves, roads, or bridges.

Much of the research regarding flood loads is based on the analysis and testing of individual components such as columns or walls subjected to stillwater, current, wave impact, or debris impact. As many buildings are elevated on piles, columns, or shear walls, the component testing is directly applicable to buildings and other structures within the flood plain. For complicated geometries and structures with enclosures below the design flood elevation, the designer should review recognized literature and consider whether project specific studies are warranted.

Flood resistant perimeters typically have openings that are sealed by deployed barriers prior to a storm event. These barriers are not considered temporary elements even if not permanently attached to the flood-resistant perimeter. All components of a perimeter flood-resisting system, including deployed opening barriers, are considered as associated components of a building and are subject to the applicable loads as described in this chapter.

Much of the impetus for flood-resistant design has come about from the federal government sponsored initiatives of flood-damage mitigation and flood insurance, both through the work of the US Army Corps of Engineers (USACE) and the National Flood Insurance Program (NFIP). The NFIP is based on an agreement between the federal government and participating communities that have been identified as being flood prone. The Federal Emergency Management Agency (FEMA), through the Federal Insurance and Mitigation Administration (FIMA), makes flood insurance available to the residents of communities provided that the community adopts and enforces adequate floodplain management regulations that meet the minimum requirements. Included in the NFIP requirements, found under Title 44 of the US Code of Federal Regulations (FEMA 2020), are minimum building design and construction standards for buildings and other structures located in special flood hazard areas (SFHAs).

Special flood hazard areas are those identified by FEMA as being subject to inundation during the 100-year flood. SFHAs are shown on flood insurance rate maps (FIRMs), which are produced for flood-prone communities. SFHAs are identified on FIRMs as Zones A, A1-30, AE, AR, AO, and AH, and in coastal high hazard areas as V1-30, V, and VE. The SFHA is the area in which communities must enforce NFIP-compliant, flood damage-resistant design and construction practices.

Answers to specific questions on flood-resistant design and construction practices may be directed to the mitigation division of each of FEMA's regional offices. FEMA has regional offices that are available to assist design professionals.

Buildings and other structures constructed in areas prone to flooding may additionally be subject to the requirements of ASCE 24 Flood Resistant Design and Construction, where required by the Authority Having Jurisdiction. ASCE 24 is referenced by the International Building Code. Minimum elevation requirements for structures set by ASCE 24 may differ from the elevations of loads in this chapter and do not preclude the occurrence of flood loads specified by this chapter. Designers may choose to elevate a structure above the elevations mandated in ASCE 24 to reduce the loads applied to the structure.

Buildings and other structures constructed in areas prone to flooding are additionally subject to the flood extent and elevation requirements of the Authority Having Jurisdiction.

C5.2 DEFINITIONS AND SYMBOLS

In ASCE 7-22 Supplement 2, higher return periods for flood design loads were introduced that exceed the base flood in elevation and spatial extent. The increase in spatial extent is limited to the addition of mapped Shaded X zones (the 500-year floodplain) for Risk Categories II, III, and IV structures.

Many communities have elected to regulate to a flood standard higher than the minimum requirements of the NFIP. Those communities may do so in a number of ways. For example, a community may require new construction to be elevated a specific vertical distance above the base flood elevation (this is referred to as “freeboard”); a community may select a lower frequency flood as its regulatory flood; or a community may conduct hydrologic and hydraulic studies, upon which flood hazard maps are based, in a manner different from the Flood Insurance Study prepared by the NFIP (e.g., the community may complete flood hazard studies based upon development conditions at build-out, rather than following the NFIP procedure, which uses conditions in existence at the time the studies are completed; the community may include watersheds smaller than 1 mi^2 (2.6 km^2) in size in its analysis, rather than following the NFIP procedure, which neglects watersheds smaller than 1 mi^2 (2.6 km^2). Flood Insurance Studies (FIS) are official reports prepared by FEMA and typically correspond to several FIRMs within a geographic region and provide supporting technical data. Flood Insurance Studies are one example of a flood hazard study that may be adopted by a community. Flood Insurance Studies typically contain flood elevation data in

C5.3 DESIGN REQUIREMENTS

ASCE’s comprehensive flood hazard requirements are split between two standards: ASCE 7 and ASCE 24, both of which are referenced by the International Building Code. While the provisions of ASCE 7 focus solely on flood loads and where flood loads are to be applied, Sections 5.3.9 and 5.3.10 reference the requirements of ASCE 24 for the application of several ASCE 7 provisions. In addition, it is helpful to designers to clarify the role of ASCE 24 and how the two documents are related, as they are on different update cycles.

ASCE 24 references ASCE 7 for loads on structures within Flood Hazard Areas. ASCE 24 provides minimum requirements for flood resistant design and construction of structures that are subject to building code requirements and that are located, in whole or in part, in Flood Hazard Areas as defined by the Authority Having Jurisdiction. ASCE 24 has requirements that vary depending on flood zone, and that govern the design and construction of foundations, elevation of the lowest floor, elevation of utility equipment, enclosures below the Design Flood Elevation (DFE), use of flood-damage-resistant materials, wet and dry floodproofing, and other miscellaneous items. ASCE 24 is adopted by reference in the International Building Code. A summary of the ASCE 24 requirements is contained on the section of FEMA’s website for flood building codes; https://www.fema.gov/sites/default/files/2020-07/asce24-14_highlights_jan2015.pdf.

Revisions to Chapter 5 in ASCE 7-22, Supplement 2 provide a risk-informed basis for flood loads with MRIs that are higher than those specified by the NFIP and provide a basis for determining flood loads for structures in areas for which the NFIP has no flood maps or requirements.

C5.3.1 Flood Hazard Area

Beginning with ASCE 7-22 Supplement 2, the Flood Hazard Area was increased from the 100-year flood plain (the Special Flood Hazard Area) to the 500-year flood plain (the SFHA *and* the shaded X-zone) for Risk Categories II, III, and IV structures to improve the performance of these structures subjected to flood events and to meet the target reliabilities of the Standard. In addition, the flood hazard Mean Recurrence Interval (MRI) was also redefined per risk category as part of the effort to transform overall reliability of the chapter. The MRI embedded in the extent of the Flood Hazard *Area* is not the same as the MRI in used to find the Design Stillwater Flood Depth in Section 5.3.3 for the hazard. A 100-year MRI flood event has a 39% probability of being equaled or exceeded over a 50-year service period, and a 500-year MRI flood event has a 10% probability of being equaled or exceeded over a 50-year service period. The increase in flood load requirements will improve the performance of structures, and better align flood hazards with the approach to wind and other hazards. The approach of ASCE 7-22 Supplement 2 better accounts for the hazard variations by applying an increased flood depth, which applies loads distributed over a larger height, instead of increasing the force over the 100-year flood depth on the structure. It is recognized that the Risk Categories III and IV structures with MRI floods that exceed the 500-year requirement correspond to a flood zone that exceeds the mapped 500-year floodplain, which is the limit of ASCE 7-22 Supplement 2. This results in structures outside of the mapped 500-year floodplain not having design requirements where they would see flood waters in larger return period events. This was deemed acceptable due to the relatively small depths seen beyond the 500-year floodplain. The increase in design MRI for flood events provides a hazard level that will provide the target reliability when combined with the load combination factors in Chapter 2.

C5.3.2 Design Loads

Wind loads, rain loads, and flood loads may act simultaneously at coastlines, particularly during hurricanes and coastal storms. This may also be true during severe storms at the shorelines of large lakes and during riverine flooding of long duration. This chapter, along with Chapter 2, provides updated guidance on how to consider and combine various types of flood loads.

For both the Flood Hazard Area and the Design Stillwater Flood Depth, which is defined in the following section, an Authority Having Jurisdiction may have adopted flood criteria that are more restrictive than that presented in ASCE 7-22 Supplement 2.

C5.3.3 Design Stillwater Flood Depth

In implementing the MRI flood requirements in Table 5.3-1, two key concepts are acknowledged. First, specific design flood requirements of the Authority Having Jurisdiction govern and may be above the mapped 100-year stillwater elevation. Second, design flood requirements in this standard may exceed those currently defined in ASCE 24 or by an Authority Having Jurisdiction relative to lowest floor elevation. Floor elevation requirements are closely related to insurance and local zoning programs and are sometimes highly contested. A design stillwater flood depth in excess of a dry floodproofed design flood elevation might narrowly be thought of as excessive because the added design depth would imply water inundation defeating the economic loss mitigation purpose of the dry floodproofed perimeter. However, an appropriately reliable structural design also prevents structural failure, or washing away of the structural system itself, which then becomes debris for other buildings.

The design stillwater flood depth is a key parameter in establishing the flood loads; it is based on the flood elevation for the specified MRI, also accounting for the erosion of the ground surface and sea level rise.

In ASCE 7-22 Supplement 2, loads in Chapter 5 are based on the stillwater elevation. In prior editions, flood loads also were based on stillwater elevation, but the chapter referenced a DFE in some load calculations. ASCE 7-22 Supplement 2 drops the reference to the DFE.

If needed for comparison purposes, the ASCE 7-22 Supplement 2 coastal DFE can be determined in accordance with Equation (C5.3-1):

$$DFE = d_f + G_e + 0.7H_{design} \tag{C5.3-1}$$

where

H_{design} = Design wave height in ft (m) as calculated in Section 5.3.7.1,

G_e = Elevation of grade at the building or other structure inclusive of effects of erosion in ft (m), per Section 5.3.5, and

d_f = Design stillwater flood depth, in ft (m), per Section 5.3.3.

The ASCE 7-22 Supplement 2 riverine DFE is the same as the Design Stillwater Flood Elevation. The DFE calculated above is not the same DFE that is used for NFIP, ASCE 24, or other model building code purposes. Each DFE should be calculated separately per the applicable Standard for its intended purpose.

C5.3.3.1 Stillwater Elevation Determination When MRI Data Not Available

For most populated areas, flood maps are available for 100-year and more recently 500-year MRI. The design flood for Risk Categories II, III, and IV structures was set at higher return periods for closer alignment with other hazards within ASCE 7. To avoid requiring a site-specific study for each building, a method of “scaling up” the flood to larger return periods has been provided in Table 5.3-1. These factors were generated based on compiling data from flood studies with thousands of individual data points. Scale factors for all MRI values apply to the 100-year stillwater elevation and do not include the effects of relative sea level change.

The following studies were used in the development of Table 5.3-1 and discussion in C5.3.3: Cialone et al. (2015), Nadal-Caraballo et al. (2015), and Melby et al. (2020).

The term *coastal sites* refers to the source of flooding, not the site location. For example, if the hazard being considered for an inland building is hurricane storm surge from a source fed by an ocean or the Gulf of Mexico, this is considered coastal flooding and the appropriate scale factors are to be used. For coastal sites, the C_{MRI} factor was developed enveloping the data from four studies from the US Army Corps of Engineers (USACE). The study regions were North Atlantic (Virginia – Maine), Texas, Louisiana, South Florida to North Carolina and Alabama/Florida Panhandle. The following figures (C5.3-1, C5.3-2) show examples of these studies with the mean, and plus and minus standard deviation factors to scale 100-year flood elevations to longer return periods.

To establish scale factors for design, the studies were grouped into two geographic regions (Gulf Coast and Atlantic Coast). To do this, the scaling curves for each study were enveloped for the region. It is recognized by enveloping the different geographic regions that conservatism is then built into the design flood depths. In addition, the scale factors are rounded up to the nearest 0.05 value. This can be seen in Figures C5.3-3 and C5.3-4 with the compiled mean scale factors. These skewed scale factors are one reason why reductions below ASCE 7 values are allowed for site-specific studies. ASCE 7 requires the coastal scale factors apply to all coastal conditions even though they were developed based on the hazard curves for the East Coast and Gulf Coast. The hazard curves for the West Coast (i.e., Washington, Oregon, California) are significantly flatter than the values listed in Table 5.3-1 thus the values are conservative.

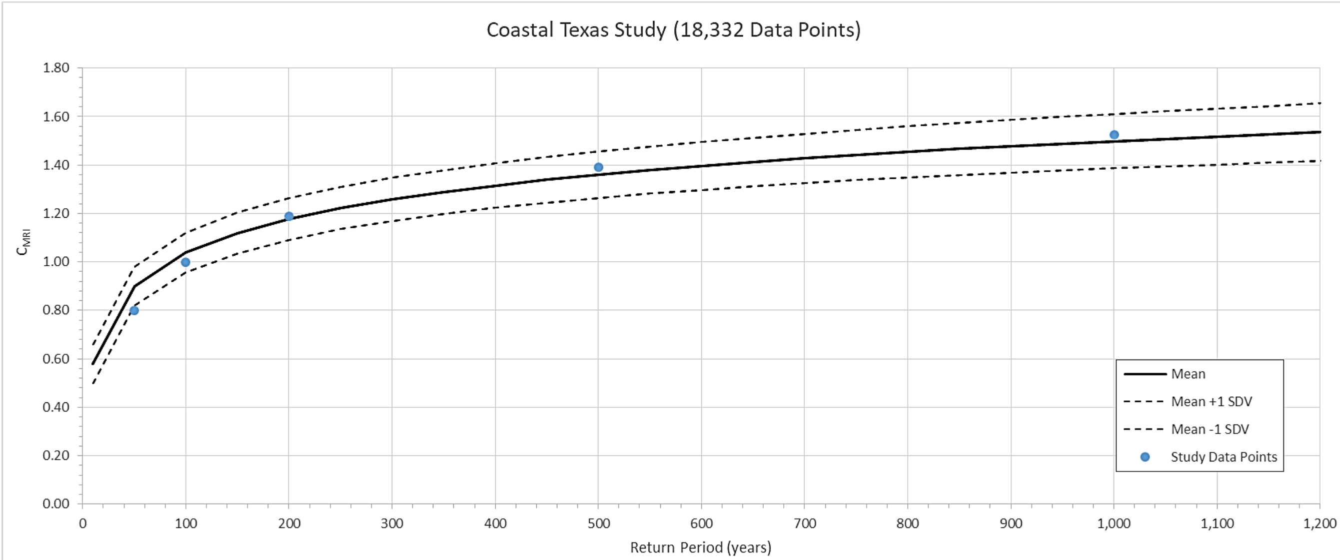


Figure C5.3-1. C_{MRI} Curve for Texas Based on USACE Data.

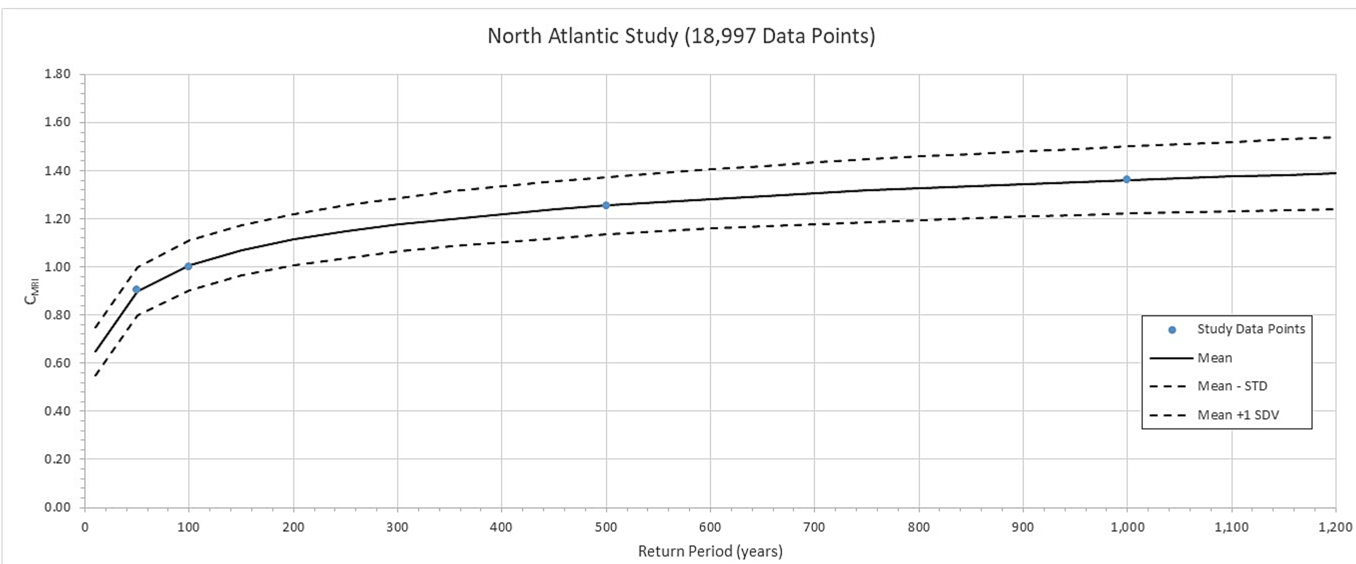


Figure C5.3-2. C_{MRI} Curve for North Atlantic Based on USACE Data.

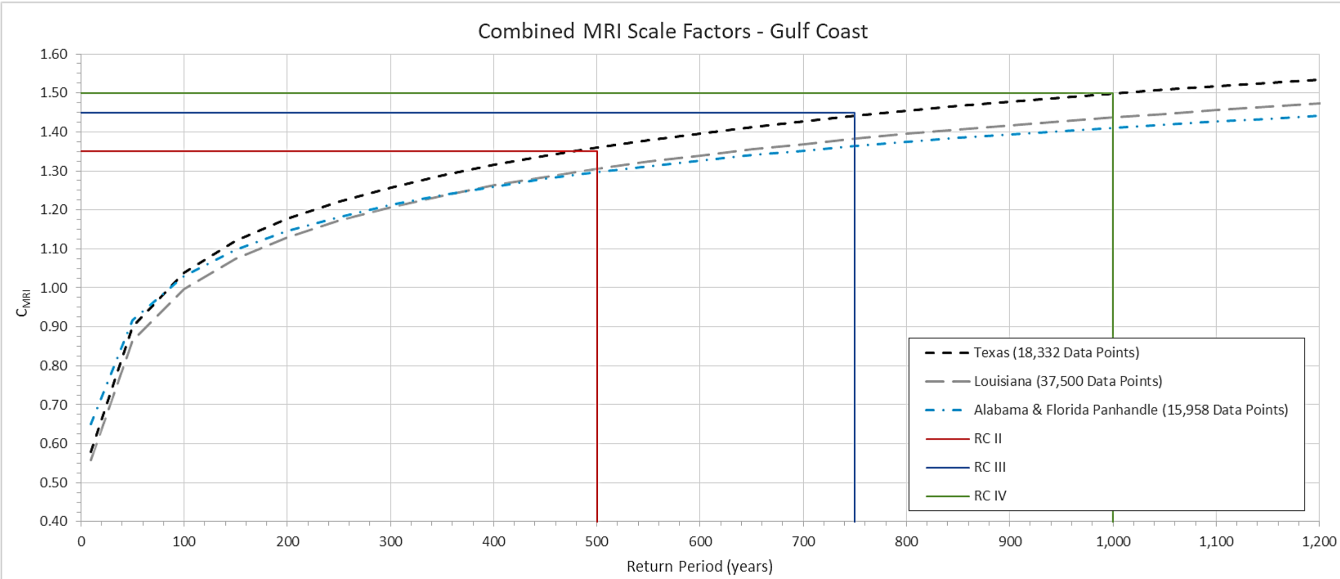


Figure C5.3-3. Combined Coastal Scale Factors for the Gulf of Mexico Coastal Region.

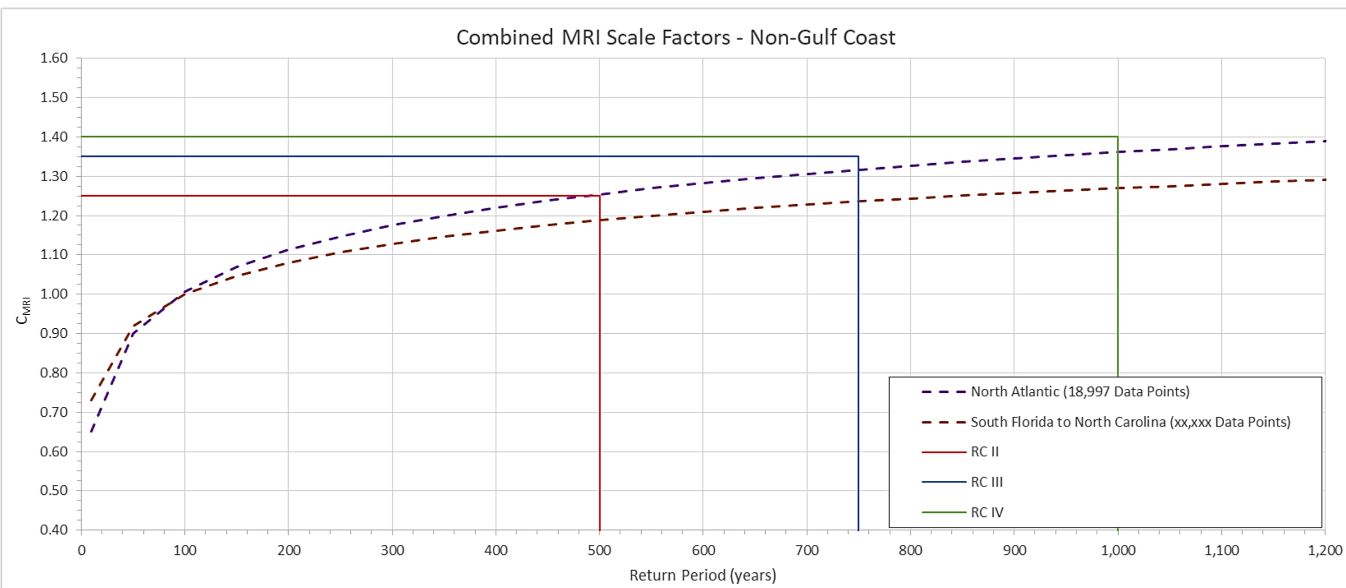


Figure C5.3-4. Combined Coastal Scale Factors for Non-Gulf of Mexico Coastal Regions.

A similar effort was done for the scale factors for the US Great Lakes. Data points from transects of USACE flood studies were taken around each lake to produce scale factor curves for each lake. Figure C5.3-5 shows the scale factor curves for each lake with the selected scale factor for each risk category.

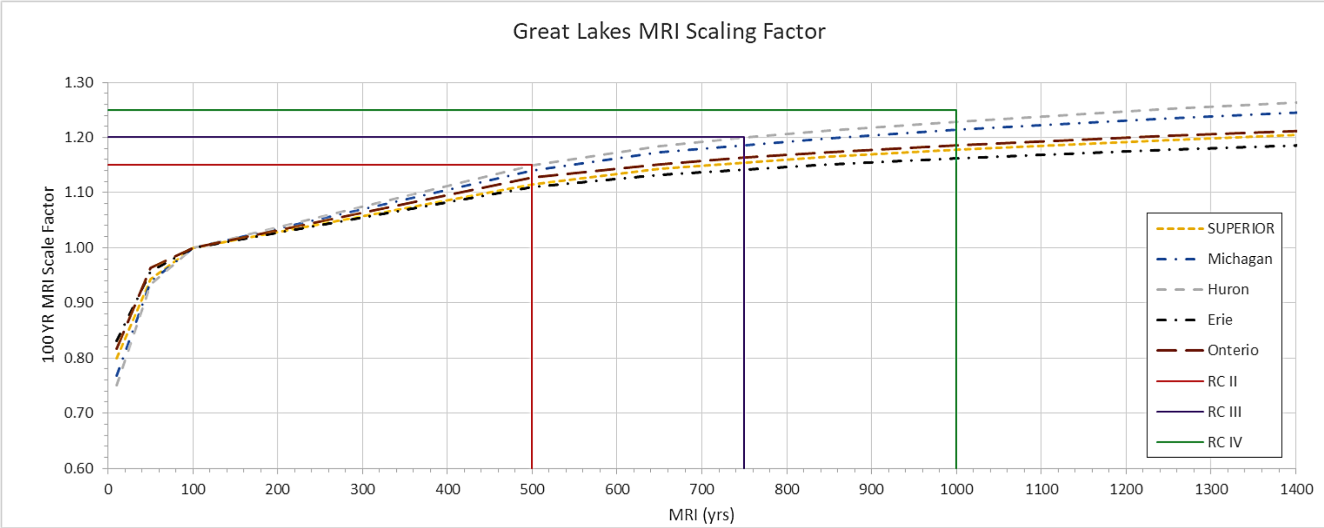


Figure C5.3-5. Scale Factor Curves for the Five US Great Lakes.

The riverine scale factors were established based on flood studies from rivers in the Midwest. Data from an Iowa floodplain study (Gilles et al.) was used to validate the theoretical scale factors. The riverine sites scale factors from location to location were significant as can be seen in Figure C5.3-6. In addition, site datum elevations were not available, so the 1-year return period had to be extrapolated; this results in the scale factors appearing larger than if the actual datum values were used.

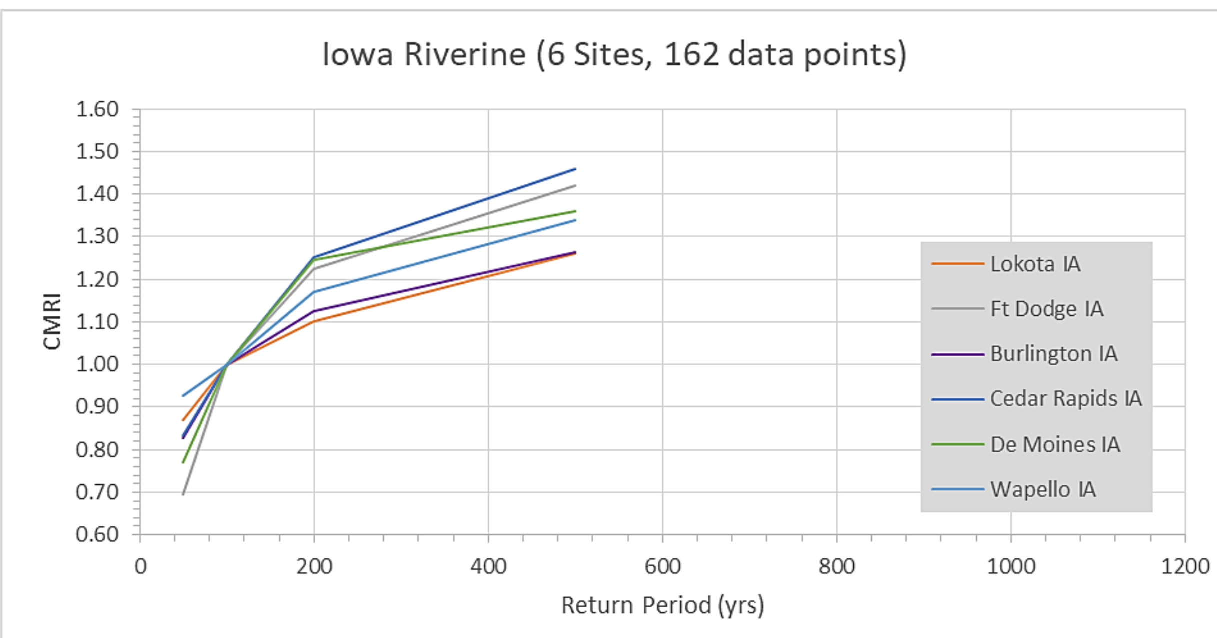


Figure C5.3-6. Scale Factor Curves for the Six Sites in Iowa.

Similar to the coastal studies, data across all sites were aggregated and a single scale factor was set for each risk category. Figure C5.3-7 summarizes the data with the scale factors shown. While not a comprehensive data set for all riverine sites, the data from six sites in Iowa provide validation of the scale factors selected.

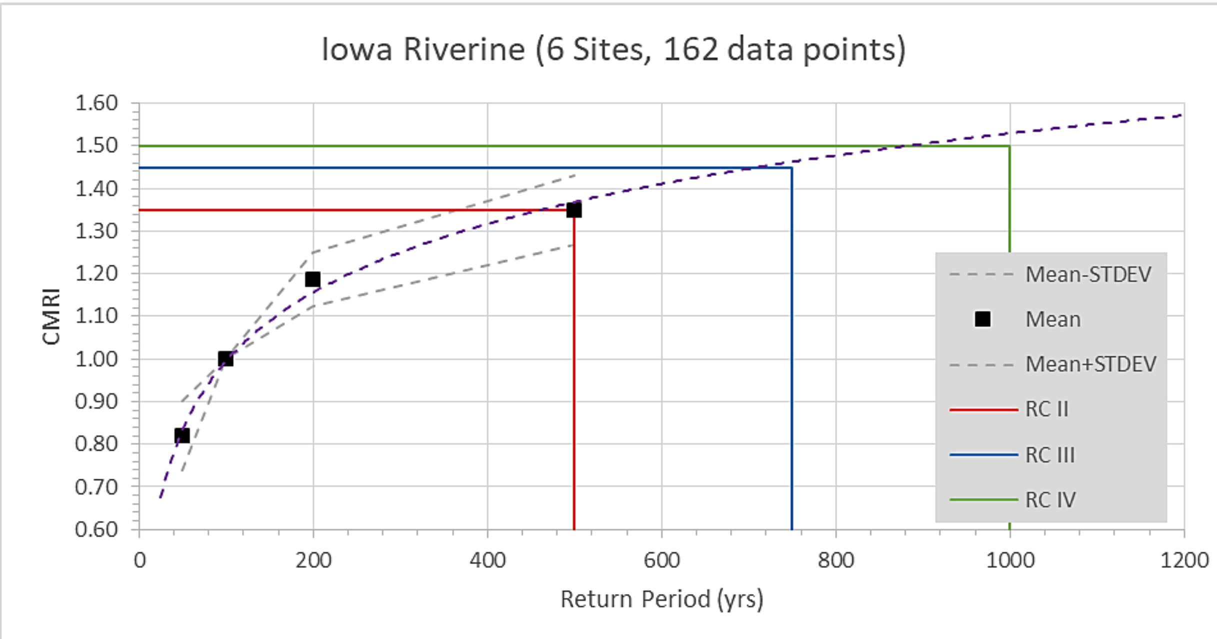


Figure C5.3-7. Summary of the Scale Factors on Iowa Riverine Sites.

The value Z_{datum} in Equation (5.3-2) is to adjust for the base water level. If Z_{datum} were not accounted for, then the elevation of the river or lake would be amplified. For most lakes, a datum is a given value for that lake and can be used for scaling. However, this is often not available for rivers where the base datum is almost never known from location to location. To simplify the analysis, the annual high-water elevation can be used to determine the flood elevations. The annual high-water level is slightly larger than the river datum, so it generates a slightly conservative flood elevation. The reference datum for each US Great Lake is shown in Table C5.3-1.

Table C5.3-1. Great Lakes Chart Datum.

Lake	Datum – NAVD88 (ft)
Superior	601.0
Michigan	578.1

Huron	578.1
Erie	570.1
Ontario	244.0

ASCE 7-22 Supplement 2 continues to recognize the FEMA Flood Insurance Rate Maps (FIRMs) as the standard for defining the Base Flood Elevation (BFE) because of their widespread coverage and usage. However, one limitation of the maps is that they do not account for sea level rise, increased precipitation, or other conditions such as long-term erosion. In addition, a site that is in the X-Zone today could be in an A-Zone in the future due to the impact of changes in the local climate between updates to ASCE 7 or to FEMA FIRMs. For this reason, structural flood load requirements have been added for areas identified in the 500-year flood zone (0.2% chance of annual exceedance) and related to the adjacent BFE.

C5.3.4 Effects of Relative Sea Level Change

Relative sea level change combines changes in sea level with the effects of local subsidence (which may be caused by extraction of underground resources, drought, and/or soil decomposition) and post-glacial rebound (caused by relaxing of land masses due to melting of glaciers at the end of the last glacial period). Several climate scenarios have been developed by the US Army Corps of Engineers (USACE), National Oceanic and Atmospheric Administration (NOAA), US Global Change Research Program, and the Intergovernmental Panel on Climate Change (IPCC), among others. In addition, many state and local climate change reports exist and may be used as guidance.

Several assumptions are needed to forecast relative sea level rise through a project's service life; thus, significant uncertainty exists in its prediction. The lower bound of future relative sea level change is typically taken as a straight-line projection of historic/recent relative sea level change. This represents the minimum required value to be used in design as defined in ASCE 7-22 Supplement 2. In lieu of more specific local or client guidance, the USACE Sea Level Change Curve Calculator (https://cwbi-app.sec.usace.army.mil/rccslc/slcc_calc.html) may be used to estimate future relative sea level change. The tool lists the historical annual rate of sea level rise or fall, and this should be projected over the anticipated lifespan of the structure. Where this historic rate of change is negative, relative sea level change is required to be taken as zero in the calculation of flood loads and conditions. Another reference that can be considered is the ASCE Manual on Engineering Practice No. 140, *Engineering Practice, Climate-Resilient Infrastructure: Adaptive Design and Risk Management* (2018).

C5.3.5 Erosion

The term *erosion* indicates a lowering of the ground surface in response to a flood event, or in response to the gradual recession of a shoreline primarily due to sediment transport. The term *scour*, as used elsewhere in this chapter, indicates a localized lowering of the ground surface during a flood, due to the interaction of currents and/or waves with a structural element. Erosion can affect the stability of foundations and increase the local flood depth and flood loads acting on buildings and other structures. For these reasons, erosion should be considered during load calculations and the design process. Design professionals often remove or mitigate effects of erosion by increasing the depth of foundation embedment, providing armoring or siting buildings away from receding shorelines (building setbacks).

Causes of erosion include storm surge, wind, relative sea level change, dredging, vegetation clearing, river damming, and land reclamation. Erosion may result in permanent or frequent inundation, more frequent flood damage, disconnection of road access networks, and increases or shifting in flood zones. Resources available to estimate current and future erosion rates include state coastal programs, USGS or State Geological Survey Agencies, USACE, and universities. Consult the Authority Having Jurisdiction for any adopted erosion projections.

Erosion can occur over different time intervals whether it be years, months, or weeks due to different modes of sediment transport. Erosion can also be episodic, occurring during a single flood event occurring over days or hours.

C5.3.6 Flood Velocity

Accurate estimates of flow velocities during flood conditions are very difficult to make, both in riverine and coastal flood events. Unlike high water marks that are often recorded during flood events, there are relatively few reliable observations of flood velocity. Potential sources of information regarding velocities of floodwaters include local, state, and federal government agencies and consulting engineers specializing in coastal engineering, stream hydrology, or hydraulics.

Figure C5.3-8 shows an example flood hazard study of the overland flow velocity due to hurricane surge at Galveston, Texas. The figure relates the estimated velocity to the local flow depth via the equation $V = \sqrt{(gh)}$ where h is the local flow depth in this figure. Figure C5.3-8 shows that this equation used in ASCE 7-16 is overly conservative (solid line) and that the use of a reduction coefficient $C_V = 0.5$ (dashed line) and upper limit ($V_{max} = 10$ ft/s, 3.05 m/s) is more reasonable.

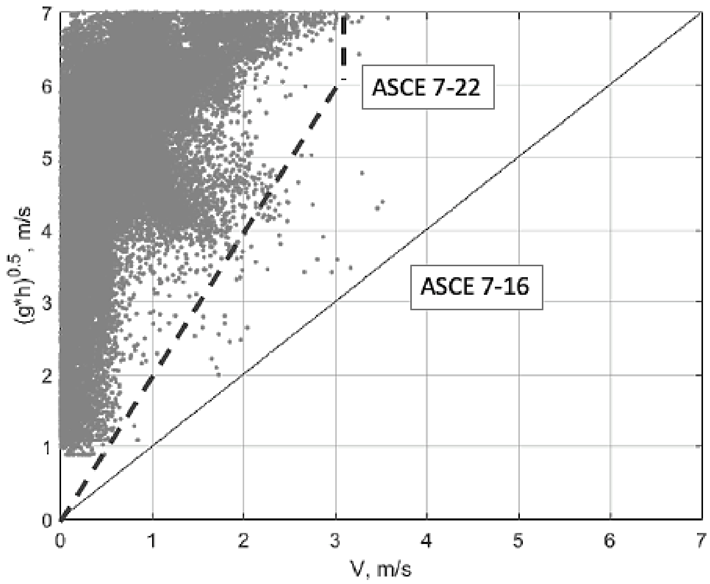


Figure C5.3-8. Example USACE Comprehensive Coastal Study Using Numerical Simulation of Overland Flow Velocity due to Hurricane Surge at Galveston, Texas.

The magnitude and direction of the flow can vary in time at a given location, and the velocity can vary spatially at a given instance in time. The maximum velocity often does not occur at the same time as the maximum flood depth. The velocity can vary with depth, with larger velocity at the mean water level and smaller velocity near grade level.

The velocity can vary with depth at a given location. Most numerical reports provide the flow as a depth-averaged velocity.

The magnitude and direction of the flow can be influenced by the presence of large buildings, such as enclosed structures of concrete, masonry, or structural steel construction located in close proximity to the site, that can accelerate the flow between buildings. Some buildings or other obstacles such as coastal forests can provide shielding to reduce the flow speed. The magnitude and direction of the flow can be affected by local changes in topography and changes in surface roughness such as pavement and vegetation.

During a coastal flood event, the flow velocity can approach a structure from different directions over the course of the event.

C5.3.7 Wave Effects

C5.3.7.1 Wave Height

Wave load and scour calculations in ASCE 7-16 depended on the initial computation of the wave height. The wave height computations in ASCE 7-16 resulted from the assumptions that the waves are depth-limited and that waves propagating into shallow water break when the wave height equals 78% of the local still water depth. Designers should be aware that wave heights at a particular site can be less than depth-limited values in some cases (e.g., when the wind speed, wind duration, or fetch is insufficient to generate wave large enough to be limited in size by water depth, or when nearby objects dissipate wave energy and reduce wave heights).

Because the assumption of depth-limited waves may be overly conservative in some cases, ASCE 7-22 Supplement 2 allows for the computation of scour depth and wave loads based on nonbreaking waves of a wave height lower than the depth-limited wave height. Figure C5.3-9 illustrates a procedure to utilize information that may yield a wave height lower than the conservative estimate of a depth-limited breaking wave at the site and a procedure to determine if the computed wave height at a site is a breaking or nonbreaking wave. The figure is divided into three branches. To start, the designer must determine if the wave height information is available at the site from a flood hazard study. If the answer is yes, then the designer proceeds to Branch 1. If no, then the designer must determine if the wave height information is available at the shoreline. If yes, then the designer proceeds to Branch 2. If no, then the designer proceeds to Branch 3.

In Branch 1, the designer determines whether the wave height at the site is provided in terms of the controlling wave height, H_c. If no, the designer converts the given wave height to the controlling wave height using appropriate wave statistics. For example, the significant wave height can be converted to the controlling wave height through the relationship, $H_c = 1.6\ H_s$, under the assumption of a Rayleigh distribution of wave heights. Next, the designer scales H_c for the desired recurrence interval corresponding to the given risk category of the structure by C_{HC} from Table 5.3-3. Finally, the designer checks whether the computed $C_{HC}H_c$ exceeds the depth-limited breaking wave height H_b. If the computed $C_{HC}H_c$ exceeds H_b, then the wave is considered a breaking wave and the design wave is set at $H_{design} = H_b$. If H_c is less then H_b, then the wave is considered a nonbreaking wave and the design wave height is set at $H_{design} = C_{HC}H_c$.

In Branch 2, the designer determines whether the wave height at the shoreline is provided in terms of the controlling wave height, H_c. The steps are the same as in Branch 1 but applied to the wave heights at the shoreline. Because the wave height is at the shoreline and not at the site, Branch 2 requires an additional step to take the wave height from the shoreline to the site, with potentially smaller wave heights at the site.

There are four methods to calculate the wave height at the site. In Method 1, the designer can use a one-dimensional transect model such as WHAFIS that is used in FEMA Flood Insurance Studies. WHAFIS is based on the 1977 National Academy of Science (NAS) study titled “Methodology for Calculating Wave Action Effects Associated with Storm Surges.” Using the methods presented in the NAS study, a designer could use Table C5.3-2 to transform the wave height from the shoreline to an inland site. Table C5.3-2 provides example wave height factors. The wave height factors in Table C5.3-2 uses the experimental results and NAS procedure, and accounts for some conservatism related to incident wave direction and building array geometry (staggering of buildings within the array).

Table C5.3-2. Number of Shielding Rows of Structures and Wave Height Factor.

NUMBER OF SHIELDING ROWS*	WAVE HEIGHT FACTOR
0 or 1	1.0
2 or 3	0.7
4 or 5	0.5
6 or more	0.3

* Requirements to be considered a shielding row:

1. A seaward shielding structure must be robust enough to remain intact during the design storm event. A building whose façade fails while the primary frame does not, is not considered a shielding structure. Buildings or barriers constructed of wood, light gage steel, aluminum, or any similarly performing material, shall be considered nonshielding unless proven otherwise by an engineering analysis.

2. Only rows less than 70% open distance between buildings relative to total distance measured parallel to shore can be considered as a shielding row.

In Method 2, the designer can use H_c at the shoreline as H_c at the site as a conservative approach unless there is a large fetch between the shoreline and the site. In Method 3, the designer can use a more advanced numerical procedure. In Method 4, the designer can use a laboratory study. Finally, the designer checks whether the computed H_{design} at the site exceeds the depth-limited breaking wave height H_b similar to the steps for Branch 1.

In Branch 3, the designer does not have access to or does not have the means to use the wave height information at the site or at the shoreline. The designer assumes a depth-limited breaking waves H_b at the site. This wave height is used as the basis for design, $H_{design} = H_b$, and is consistent with the ASCE 7-16 standard. Branch 3, consistent with ASCE 7-16, is the most conservative approach and is permitted to be used.

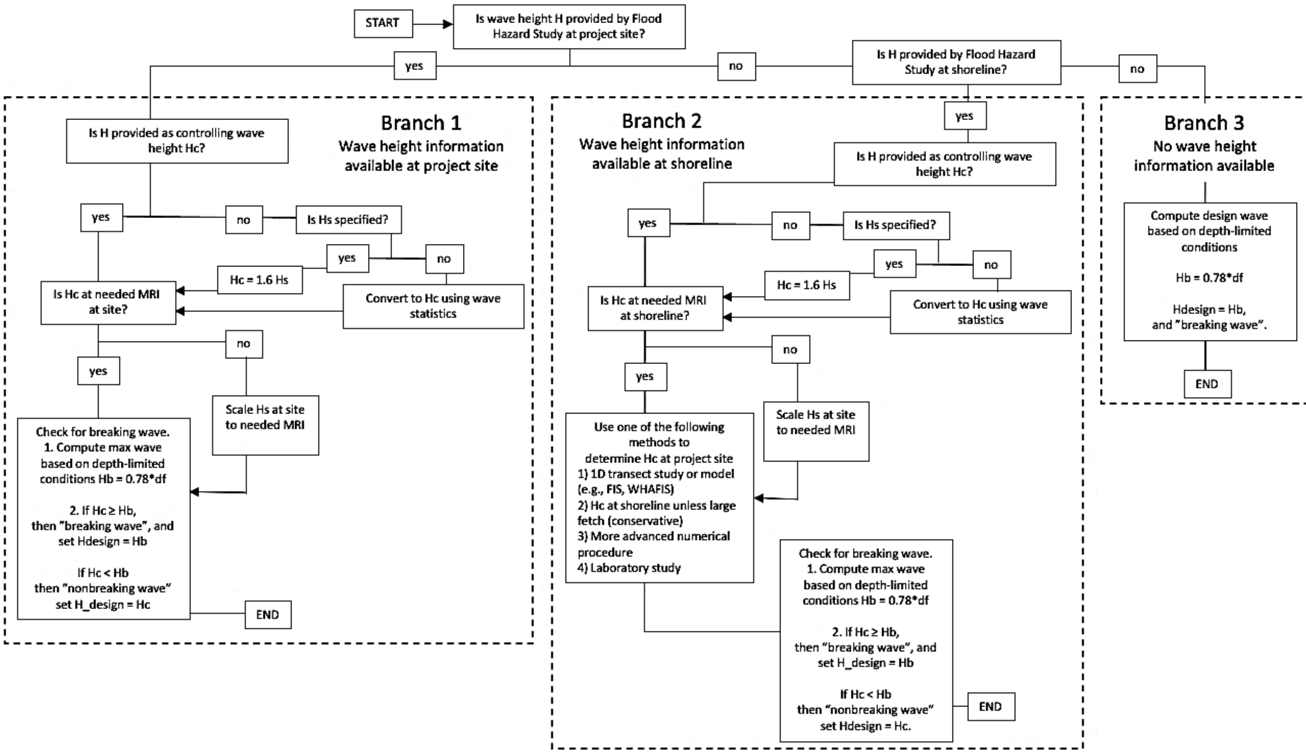


Figure C5.3-9. Procedure to Determine Wave Height at Site for Breaking and Nonbreaking Waves.

C5.3.7.2 Wave Period and Wavelength

Equation (5.3-9) can be used to estimate the wave period based on the design wave height. However, designers should consider additional sources to estimate the wave period such as a flood hazard study for that area. Designers should be aware that the wave periods from hurricanes at a coastal site exposed to the ocean can be in the range of 12 to 20 s.

Equation (5.3-10) relates the wavelength to the local water depth and wave period. For a local water depth of 16.4 ft (5 m) and for wave periods in the range of 12 to 20 s, the wavelength is in the range 269 ft (80 m) to 441 ft (134 m).

C5.3.8 Scour

The term *scour* indicates a localized lowering of the ground surface during a flood due to the interaction of currents and/or waves with a structural element. Scour is a complex mechanism related to hydrodynamic forces that causes local increases in bed shear stresses and the resulting sediment transport. Clays, silts, sands, and gravels are generally considered erodible when they can be moved under the design velocities. Paved surfaces, rock formations, and armored protection specifically designed for scour prevention are considered non-erodible when they cannot be moved under the design velocities. Vegetation is sometimes used as a means of stabilizing land and protecting against scour. However, the level of protection against scour is variable depending on the flood conditions, the elevation and slope of the planted surface, and the type, size, and density of the vegetation provided. If vegetation is being considered as part of scour protection, it should be maintained and carefully evaluated for its ability to stay in place during flooding, and to resist flood loads up to and during the design flood conditions.

While scour is a function of soil particle size, waves, and currents, the predictive equations for scour depth in general are empirically derived from small-scale laboratory tests and are relatively simplistic. Scour theory and studies for various coastal structures are discussed in depth in the *Coastal Engineering Manual* (USACE 2002). ASCE 24 requires that foundations extend to a depth sufficient to provide support to the structure taking into account the effects of scour. Scour is considered for evaluation of unbraced pile length, pile capacity, and review of undermining of foundations and walls. Unlike the effects of erosion, local scour does not affect the calculation of the design flood depth at a site or increase the wave height.

The methodology for scour calculation in this section is derived from USACE (2002) specific to coastal sites. Scour at riverine sites is different from coastal sites and is a common aspect of bridge foundation design. FHWA provides an industry standard resource for evaluating scour at bridges (2012) that can be adapted for scour calculations at riverine sites. Because the FHWA standard is intended for bridges, adaptation of these methods for buildings should be used with caution by the designer.

C5.3.8.1 Scour at Walls

The scour depth prediction methods are based on the *Coastal Engineering Manual* (USACE 2002) in EM 1110-2-1100 – Part VI, Change 3. The methods assume the erodible bed is composed of noncohesive sediment. Methods for predicting scour depth of cohesive soil are virtually nonexistent; however, estimates for scour of noncohesive erodible bed are expected to provide conservative estimates of the scour depth.

Part VI of the *Coastal Engineering Manual* (USACE 2002) discusses scour for nonbreaking wave using the results from 12 movable-bed model tests and concludes that uniform, regular nonbreaking waves produce a repeating pattern of scour and deposition as a function of distance from the toe of the vertical wall. For fine sand, the maximum scour nearest the wall occurs at a distance of $L/4$ from the face of the wall, where L is the wavelength. Waves with a relatively short period (2 to 3 s) in shallow floodwater depths [10 ft (3.05 m)] can have wavelengths between 20.4 ft (6.2 m) to 41.8 ft (12.7 m), in which case, the scour due to nonbreaking waves can occur close to the wall and may undermine the soil support. Hence the standard conservatively requires the scour to be considered at the face of the wall. Scour due to breaking waves occurs at the face of the wall and is generally greater than the scour depth for nonbreaking waves.

C5.3.8.2 Scour at Vertical Piles and Columns

The provisions regarding vertical pilings and columns referenced in this section are based on small diameter piles (such that the pile diameter is less than one-tenth of the incident wavelength), and as such do not significantly affect the incident wave. The pilings and columns for buildings and other non-bridge structures generally fall in this category. Recommendations for large diameter piles can be found in the *Coastal Engineering Manual* (USACE 2002).

Scour has been observed to extend deeper for pile groups in coastal buildings. In particular, low-lying buildings with predominantly silty soils at grade are most susceptible to pile group scour. The interaction of waves and currents on pile groups is dependent on flow characteristics, wave conditions, pile size, and spacing. The potential for group effects in scour depth calculation should be evaluated based on available historical records and factors affecting pile scour. An estimate for scour around a pile group is given in Chapter 8 of FEMA P-55 (FEMA 2011), based on field measurements taken after flood events.

C5.3.9 Debris

Section 5.3.9 establishes the design requirements relating to debris effects. There are two types of debris effects considered in these provisions: debris damming and debris impact. Floods typically carry large amounts of objects that cannot resist the effects of flood loads and become dislodged from the ground or mooring. Floods with a depth less than 3 ft (0.91 m) are typically not capable of generating debris fields causing debris strikes or accumulation (damming) because objects drag along the ground due to the geometry of the element, and undulations in the topography. Therefore, if the site of interest has a design stillwater flood depth less than 3 ft (0.91 m), debris impact loads and debris damming do not need to be considered.

C5.3.9.1 Debris Impact

Based on the highly variable nature of water flow, with many load cases of different flood depths [including depths ranging from the design stillwater flood depth down to the threshold depth of 3 ft (0.91 m)] and flow directions, the most severe impact load location and direction must be considered. For example, in flexure this is often mid-height of a column in its weak axis. Impact loads must be applied at all locations at or below the design stillwater flood depth, with the corresponding hydrostatic and hydrodynamic loads, to determine the most severe location. It is permitted to not reduce the hydrostatic and hydrodynamic loads from the design stillwater flood depth while varying the impact load depth, but this may lead to very conservative loads.

In prior versions of ASCE 7, debris loads were only required for the 100-year flood zone. ASCE 7-22 Supplement 2 increased the design criteria to include the Shaded X-Zone (area between 100-year and 500-year flood) which increased the zone of debris requirements. However, a large portion of the shaded X-Zone has design stillwater flood depths (d_f) of less than the 3 ft (0.91 m) minimum threshold depth for debris, which will exempt much of the Shaded X-Zone from debris impact loads. Debris impacts are rare events; therefore, impact loads do not need to be considered simultaneously on multiple sides, planes, or panels of structural flood-resisting elements as the odds of multiple simultaneous debris impacts are minimal.

There are four exceptions to the debris impact requirements. Exception 1 is for when a designer has a site-specific study that shows definitive flow directions during the storm surge and recession, in this case debris impact need only be considered from the possible directions indicated in the study. The second exception occurs for riverine conditions where the flow direction is generally known, the debris strikes can be assumed to occur from the upstream direction with a +/-22.5 degree arc to account for some variation in flow. The third is an exception for one- and two-family dwellings. While flood driven debris impacts have been seen on single-family homes, this exception exists because the typical framing construction of these structures (light-framed wood) is not economical to design for a debris impact. The fourth exception is for Risk Category II structures outside the 100-year flood zone. This exception removes debris impact requirements for these structures between the 100-year and 500-year flood zones. Exceptions 3 and 4 only relate to debris impact requirements; design for debris damming is still required.

C5.3.9.1.1 Debris Impact Objects

It is not practical to consider all types of debris impacts for all risk categories of structures. Larger more rare debris impact strikes only need to be considered by more critical facilities that greatly affect a community's resilience. Debris objects considered in this chapter are wood poles/logs, passenger vehicles, small vessels (pleasure craft), shipping containers and larger vessels. These objects serve as proxies for the full suite of possible debris objects.

Table 5.3-4 is provided to summarize the debris types required for different structures based on risk category, threshold depth, and if façade (architectural components and deployed opening barriers) impacts are required to be checked. For all debris types the primary structural elements should be checked for debris strikes to provide life safety for the occupants of the building. However, unlike the majority of ASCE 7 provisions, whose focus is life safety, design for flood (through ASCE 24) also includes a focus on keeping habitable/occupied areas of buildings and other structures "dry." While dry floodproofing is not an explicit requirement of ASCE 7 to achieve this goal, it is critical that the façade, inclusive of opening barriers, remains intact and undeformed enough to meet permissible leakage standards, where structures are dry floodproofed. Table 5.3-4 attempts to balance these two purposes by including common, rational, debris objects for design of the watertight perimeter and rare, extreme debris objects for primary structure life-safety checks. Footnote 3 of Table 5.3-4 clarifies that the debris impact requirements apply only to nonload-bearing perimeter elements if part of a dry floodproofing system per ASCE 24. The smaller debris requirements for nonload-bearing (dry floodproof systems) is in alignment with ASCE 24 for limiting water damage in buildings by keeping the water out. The structural load bearing elements (defined as columns, walls, piles, and transfer beams) are checked for additional larger debris load types in addition to the smaller ones. These structural elements have been selected as elements critical for vertical stability of the structure to resist global collapse.

C5.3.9.1.2 Site Hazard Assessment for Localized Marine Debris, Shipping Containers, Ships, Small Vessels, and Barges

Large debris such as shipping containers and large vessels are often found in medium to large ports. These ports are often long distances from most significant structures. Even in extreme events these large debris objects have been observed to only travel limited distances.

Conversely, based on observations from previous events (e.g., Hurricane Katrina) large debris objects (shipping containers, ships, etc.) that start in an open area (i.e., harbor) can travel significant distances over open land/water but are limited in their travel over developed environments. Based on these field observations a two-step process was developed to determine the area of impact for large debris types.

Due to the observed travel sometimes exceeding 2 mi (3.2 km) over open land (FEMA 2006), the designer must first consider a 10,000 ft (3.03 km) radius from the source of the debris. (Note: for large debris sources, the radius should be taken from the worst-case edge of the source) over water or undeveloped land (i.e., water, beach, roads, open lots). From any point within that radius a second radius for travel into urban environments should be taken with a distance per Table 5.3-5. The travel distance is based on two factors: level of development (Moderate or Heavy) and the types of debris (shipping containers or ships/barges). A more developed urban environment will restrict the travel of debris; therefore, the radius considered is much smaller [500 to 2,000 ft (152 to 610 m)]. The definition of Moderate versus Heavy density is developed based on scaling of typical developed urban and suburban environments for building densities. When calculating densities, only structures that are tall enough (defined as 75% of flood depth) should be considered. In most flood zones, a one-story structure will have adequate height to meet this requirement. Additional information about debris transport under hurricane surge can be found in Kennedy et al. (2020).

An example of the site hazard assessment using a shipping container yard in Gulfport, Mississippi, is shown in Figure C5.3-10 and a second example with a container yard on an inland river is shown for Charleston, South Carolina, in Figure C5.3-11.

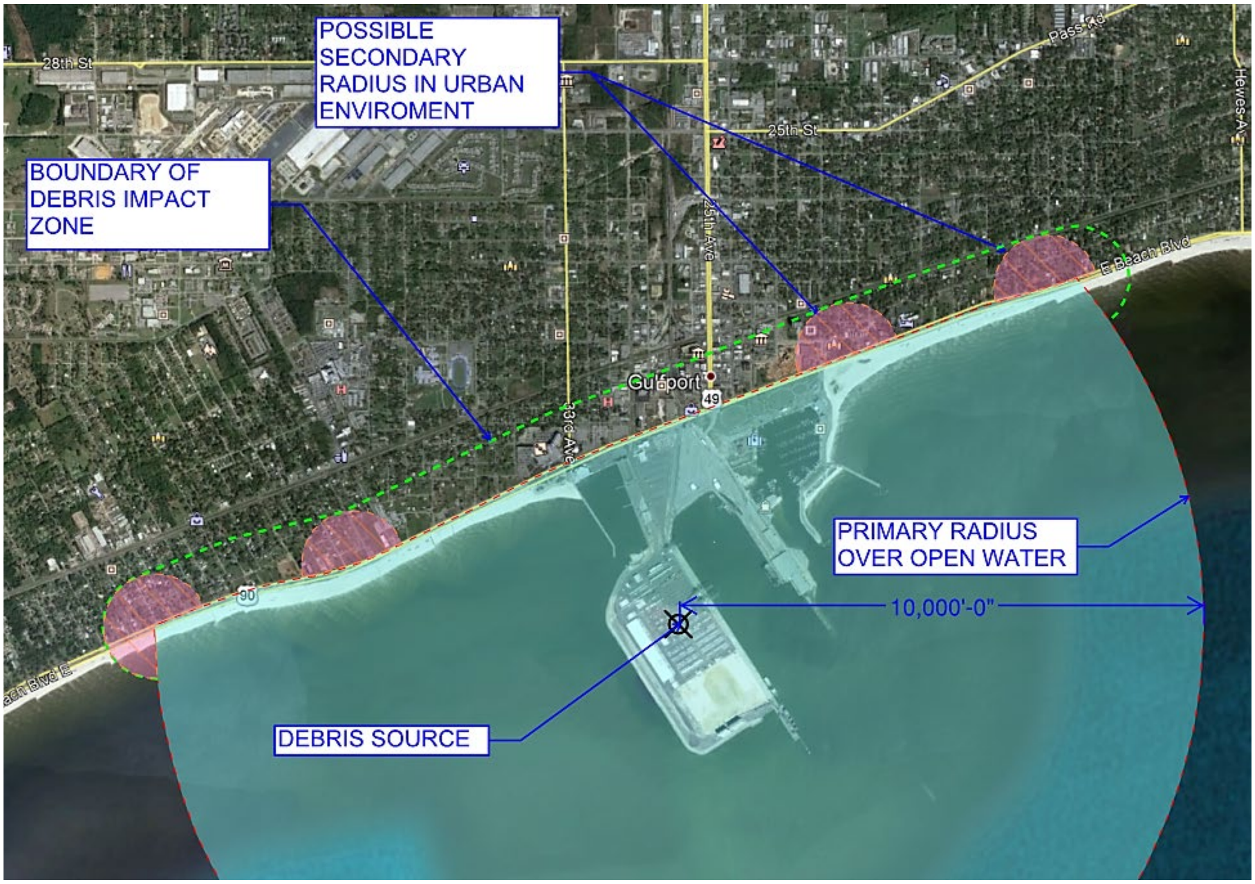


Figure C5.3-10. Example of Site Hazard Assessment for Container Yard in Gulfport, Mississippi.

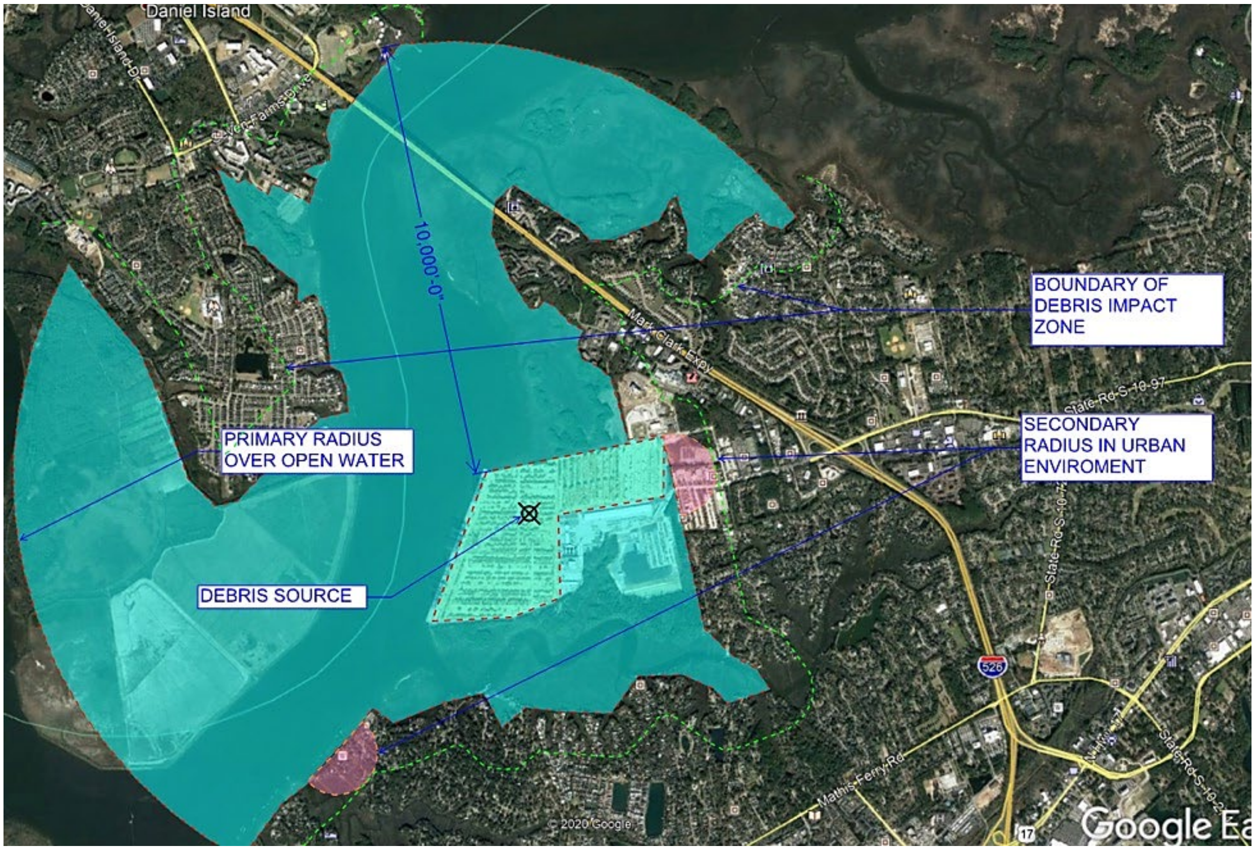


Figure C5.3-11. Example of Site Hazard Assessment for a Container Yard in Charleston, South Carolina.

Debris can become grounded based on lack of sufficient flow depth over local topography or substantial structures that would survive a design flood. Therefore, grounding may be considered to limit debris movements. For a debris type, if no viable path between the source of the debris and the site where the flood depth is greater than the threshold depth in Table 5.3-4, the debris type does not need to be considered.

ASCE 7-16 (and previous versions) of the debris loading commentary carried a blockage coefficient which talked about reduction in velocity of debris based on "screening and sheltering." The intent of the current exception is if a design using rational methodology can show that a debris object cannot reach the building, design for this debris object can be neglected.

C5.3.9.1.3 Extraordinary Debris Impact Loading

Consideration of extraordinary debris is only required for Risk Category IV structures with a design stillwater flood depth of 12 ft (3.7 m) or greater. This provision is intended to protect the most critical buildings with the largest community resilience hazard. The designer is required to consider the largest vessel in the area (determined by the project site). Instead of directly designing for a vessel of this size, which is likely not economically practical, the designer is required to consider the loss of a structural element (i.e., a column) in a progressive collapse analysis.

C5.3.9.2 Debris Damming

Debris damming is the effect of debris becoming entangled with open structures, which causes increased drag forces on individual elements and the entire structure. Structures near the coast often have foundations that consist of individual columns/piles up to the first floor level; the spacing of these piles can vary from tightly spaced less than 10 ft (3.05 m) to larger spans greater than 30 ft (9.15 m). The ability of debris to start the damming between columns is a function of the spacing between the columns and the size and type of debris. Figure 5.3-1 provides a closure ratio that the engineer is required to use to calculate the hydrodynamic drag, which is based on the clear spacing of these vertical elements perpendicular to the flow. The closure ratio is defined as the fraction of space between the columns that needs to be considered as closed for design of these vertical elements for hydrodynamic loading. In addition, for open structures or structures with breakaway walls per Section 5.3.10 the debris damming needs to be considered for the design of the lateral force resisting system. Because the debris is surrounded by water, the debris dam does not impart additional hydrostatic loads in the horizontal direction. At vertical elements with clear spacing of 10 ft (3.05 m) and less, the closure ratio is 0.7 (i.e., 70% of that opening must be considered closed). The lower threshold of 10 ft (3.05 m) clear was set based on the debris types that are likely to be able to cause significant damming (i.e., cars). These objects exceed the 10 ft (3.05 m) clear distance in length but the likelihood of a 16 ft (4.9 m) car getting stuck on adjacent columns goes down as the column clear spacing exceeds 10 ft (3.05 m). At vertical elements with clear spacing of 30 ft (9.15 m) and greater, the probability of substantial debris being caught against the structure is minimal.

The spacing where damming effects can be neglected was established by review of post-hurricane field studies. The 30 ft (9.15 m) span was chosen because it exceeds the length of most debris types such as logs, cars, and small vessels which could cause the damming. The limit of 70% closure was established as the maximum for the closure ratio based on observations of debris damming in flood events. The requirement for a structure to consider two adjacent bays of debris damming ensures that a given column or wall sees a proper flood drag load due to damming on either side. The minimum requirement of two bays or 50 ft (15.2 m) is set based on the probability of seeing a large width building with debris accumulating over the entire width, which could unnecessarily increase the demands on the lateral force resisting system. The section points the designer to Section 5.4.3 for use of the closure ratio in the hydrodynamic drag forces section.

C5.3.10 Loads on Breakaway Walls

Floodplain management regulations require buildings in coastal high hazard areas to be elevated to or above the design flood elevation by a pile or column foundation. Space below the ASCE/SEI 24 Design Flood Elevation (DFE) must be free of obstructions to allow the free passage of waves and high-velocity waters beneath the building (FEMA 2020). Floodplain management regulations typically allow space below the DFE to be enclosed by insect screening, open lattice, or breakaway walls. Local exceptions are made in certain instances for shear walls, firewalls, elevator shafts, and stairwells. Check with the Authority Having Jurisdiction for specific requirements related to obstructions, enclosures, and breakaway walls.

Where breakaway walls are used, NFIP regulations require that the walls meet certain prescriptive load requirements or be certified by a registered professional engineer or architect as having been designed to meet the NFIP performance requirements. Meeting the NFIP performance requirements means that the structure to which breakaway walls are attached should withstand both of the following: (1) load combinations, including flood loads acting on the structure and the breakaway walls, at the point at which breakaway occurs, and (2) load combinations for the full flood design loads, assuming the breakaway walls have detached.

The structure should be designed for the expected loads transferred from the breakaway walls and partitions to the structure. Loads on breakaway walls and partitions are usually calculated without using a material reduction factor (i.e., without phi-factor) and usually calculated assuming that actual yield or ultimate strengths are larger than the minimum strengths usually specified in new design. An approach similar to that used to compute expected strength in ASCE 41 could be used.

The NFIP prescriptive minimum breakaway load requirement of 10 psf (0.48 kN/m^2) has been adjusted in ASCE 7 to reflect Load and Resistance Factor Design methodology. Inasmuch as wind, earthquake, or lateral earth pressure loads may exceed the 100-year flood load, breakaway walls may be designed for higher loads, provided the design conditions of Section 5.3.10 are met. FEMA (2021) provides guidance on how to meet the performance requirements for certification.

C.5.3.11 Site-Specific Studies

This standard has been developed to conservatively establish the flood hazard, and therefore the associated flood loads, for buildings and other structures. Use of a site-specific study can result in a lower estimate of flood loads. Also, while the Standard mandates the flood hazard be considered from all possible directions, a site-specific study may be able to eliminate some possible directions after a full cycle of inundation and recession is considered.

Table 5.3-6 provides maximum allowable reductions from the velocity and wave height and period computed from Sections 5.3.6 and 5.3.7 for site-specific studies with and without peer review per Section 1.3.1.3.4. Velocity and wave height and period hazards can have large variability overland due to local conditions such as topography that have a greater effect on waves and currents than flood depth. Figure C5.3-8, for example, shows a large variability in the velocity simulated numerically for a given flood depth. Buildings and other obstacles can also influence the velocity and wave conditions, potentially leading to large reductions in velocity and wave height and period beyond the values in Table 5.3-6 (NAS 1977, East et al. 2008, Tomiczek et al. 2014). Velocity and wave height reductions reduce the hydrodynamic and wave loads in this chapter. Wave height and period reductions reduce the amount of scour considered at walls. However, the maximum allowable reductions provided in Table 5.3-6 are intended to provide reasonable limits that substantially reduce the potential for underestimation of velocity and wave height and period.

Site-specific studies may range from relatively simple to complex. The simplest studies use better topographic information in conjunction with hydrologic and hydraulic parameters established by the flood hazard study adopted by the Authority Having Jurisdiction. The more complex involve more accurate topographic information and re-estimation of one or more of the hydrologic and hydraulic parameters using numerical, statistical, or physical models and procedures accepted by the Authority Having Jurisdiction. Some site-specific studies may extend design flood parameters to higher MRIs than those contained in the adopted flood hazard study. Site-specific studies may account for the effects of local topography and development/land use on water surface elevations and flow fields, and in coastal areas, wave fields. Site-specific studies can reveal local variations in design flood conditions and erosion, including those resulting from channeling and sheltering by buildings and other structures.

C5.3.12 Performance-Based Design.

Performance-based design (PBD) methods are expected to yield flood design alternatives for Risk Categories II, III and IV structures for a design equivalent to the code-required minimums. There are many structures, such as multi-family housing, hospitals, assembly areas, and even hazardous chemical sites, that are in a special flood hazard area and, if undergoing substantial improvement or repair of substantial damage after a flood, might be required to be elevated to or above the DFE, and thus are faced with a decision to abandon the site or expensively renovate the structure to comply with the building code and NFIP flood regulations. Many existing buildings and some new buildings cannot feasibly be raised to comply with flood elevations, but owners might be able to improve their resilience and service to the community by selectively improving the performance of their most flood-vulnerable operational systems and/or their building access.

In addition, new commercial and industrial buildings are currently allowed to dry floodproof the entire facility and comply with the code, but they are not allowed to evaluate and choose to protect only the critical, isolated operational functions that are located within the flood depth. PBD allows this as a code compliant option if the design can demonstrate performance objectives of the facility are met. Also, new residential buildings, including multifamily buildings, are required to be elevated to be code compliant. As such, lobbies and other entrances for these structures must also be elevated in order to be code compliant. This provision will assist the owner in meeting the code if the PBD process demonstrates such spaces can meet the expected building performance if used for nonresidential purposes.

An independent peer review by qualified experts is required per Section 1.3.1.3.4 for flood PBD, similar to wind and earthquake.

The designer should help building owners evaluate the flood risk of their building and mechanical systems in conjunction with the desired performance of these systems during and after flooding. Based on the expected performance at various risk levels, the designer should determine which elevations or flood resistant design techniques will achieve the expected performance. Once the design risk level (the flood design return period) has been determined, then the designer can determine how best to achieve the selected risk level. This may be done with elevation, with floodproofing structures, with building hardening, with alternative technologies so some mechanical systems could be eliminated, or other resilience enhancing techniques.

The performance objectives should be framed by defining performance levels and then describing the expected performance of various risk category buildings when impacted by various flood hazard levels. The flood hazard levels should be defined by the Mean Recurrence Interval (MRI) in years. There are many ways to select the appropriate flood hazard level. Table C5.3-3 provides a listing of various MRI in years and their associated exceedance probabilities for three different lifespans (50 years, 75 years, and 100 years). The larger the MRI (return period), the less frequent the event but the greater the consequences of that event.

Table C5.3-3. Exceedance Probabilities.

MRI (yrs)	P (50)	P (75)	P (100)
10	99.5%	100.0%	100.0%
50	63.6%	78.0%	86.7%
100	39.5%	52.9%	63.4%
200	22.2%	31.3%	39.4%

500	9.5%	13.9%	18.1%
700	6.9%	10.2%	13.3%
1,000	4.9%	7.2%	9.5%
2,000	2.5%	3.7%	4.9%
2,500	2.0%	3.0%	3.9%

The table suggests that for a building life of 50 years, the 100-year MRI event has a 39.5% chance of being equaled or exceeded during that lifespan. For the same lifespan, the 1000-year MRI event only has a 5% (4.9%) chance of being equaled or exceeded during that lifespan. Many Risk Categories III and IV structures would expect to have a greater than 50-year life. A NIST study focused on how to approach designs for immediate occupancy (NIST 2018) and suggested there might be three hazard levels: routine (serviceability), design (affects the building), and extreme (affects the community). It is recommended that the routine or serviceability hazard level be the 100-year MRI; the design level hazard should be at least 500-years for Risk Category II structures, which would make the coastal hazards of wind and flood have similar risk levels (wind has a 700-year MRI for Risk Category II structures, a 1700-year MRI for Risk Category III structures and a 3000-year MRI for Risk Category IV structures); the hazard level for the extreme event could be an event of any frequency greater than 1000 years, as long as it is established with the community and building owners/operators as part of the stakeholder group. Minimum design flood MRIs are provided in Table 5.3-1 of the standard.

Table C5.3-4 illustrates a matrix of expected performance and hazard levels for structures in accordance with their risk category (as used in Section 1.5.1 and Table 1.5-1 of ASCE 7). For flood hazards, unlike other hazards, it is assumed that all occupants have been evacuated prior to the event for structures in Risk Categories I through III.

Table C5.3-4. Matrix of Expected Performance and Hazard Levels for Flood.

Hazard Level versus Performance	Operational	Repairable	Significant Damage	Unsafe to Occupy
Routine	RC IV, RC III	RC II		
Design	RC IV	RC III, RC II		

Extreme		RC IV	RC III	RC II

The performance levels shown in Table C5.3-4 are defined as follows:

Operational: This performance level suggests that however the building owner defines operational is how the flood design should be approached. For Risk Category IV structures the building must be operational during a routine event. This performance means that the occupants should not get wet while in the building, that the occupants should be able to travel to and from the structure just prior to and immediately after the event, and that the functional parts of the facility are operational such as water, sewer, heating, or cooling systems. For Risk Category IV structures, the operational aspects mentioned above would be required for design level events.

Repairable: This performance level suggests that the structure should be repairable after a given event which might include clean up, replacement of small elements of the mechanical or electrical systems, suggesting a reasonably short period of downtime. Repairable could also be defined as more extensive repair to portions of the mechanical or electrical systems or to the building envelope as long as the structure could safely be re-occupied. Likely there has been some water intrusion and finishes will need to be replaced locally to repair damage and inhibit mold growth. The influence of weather may need to be considered (i.e., in winter, if the heating system is not working, then occupancy may not be achievable or desirable).

Significant Damage: This performance level, also known as a “yellow tagged structure” in the ATC 45 system, suggests that there is no fear of collapse, but there was potentially significant water infiltration. Occupants should be able to return after the event to start the effort of removing debris, damaged systems, non-load bearing, partition walls, and finishes. There might be flood-borne debris that has impacted the structure, but this debris did not damage the building frame in a way that might cause a partial collapse of primary elements.

Unsafe to Occupy: This performance level, also known as a ‘red tagged structure’ in the ATC 45 system, suggests that there is significant damage to the structural frame, especially the foundation. There could be severe undermining of the footings from scour or erosion; there could be damage to the foundation walls or footings from large flood-borne debris, and the severity of the scour at the structural walls might make entry hazardous.

ASCE has quantified expected structural reliabilities for components that are necessary to provide a consistent risk-based design for the various risk category structures and for various hazard levels as defined by the MRI. These reliability targets are provided in Section 1.3.1 in ASCE 7.

The designer should follow the performance-based design procedures provided in ASCE 7 Section 1.3.1.3. There has been little research on fragility functions of structural members subject to various flood loads, so the designer should anticipate that some fragility functions will need to be developed and independently verified before using them for PBD.

C5.4 LOADS DURING FLOODING

C5.4.1 Load Basis

Water loads are the loads or pressures on surfaces of buildings and other structures caused and induced by the presence of floodwaters. These loads are of two basic types: hydrostatic and hydrodynamic. Wave loads can be considered a special type of hydrodynamic load and consist of nonbreaking or breaking wave loads. Debris loads are of two basic types: debris impact loads and debris damming loads. Debris impact loads result from objects transported by floodwaters striking against buildings and other structures or parts thereof. Debris damming loads result from the accumulation of debris on a structure subjected to flowing water.

C5.4.2 Hydrostatic Loads

Hydrostatic loads are those caused by water at rest and the resulting pressure exerted on inundated objects. Hydrostatic loads can occur either above or below the ground surface, from water that is free or confined within a structure. These loads are equal to the product of the hydrostatic pressure multiplied by the surface area on which the pressure acts.

Hydrostatic pressure at any point is equal in all directions and always acts perpendicular to the surface on which it is applied. The magnitude is proportional to depth as shown in Figure C5.4-1. Hydrostatic loads acting on inclined, rounded or irregular surfaces should be calculated based on the geometry of the surfaces and the distribution of hydrostatic pressure. Hydrostatic pressures may be used for structural calculations or may be resolved into vertical downward or upward loads (uplift or buoyancy) and lateral loads for simplified use based on the geometry of the surfaces and the distribution of hydrostatic pressure.

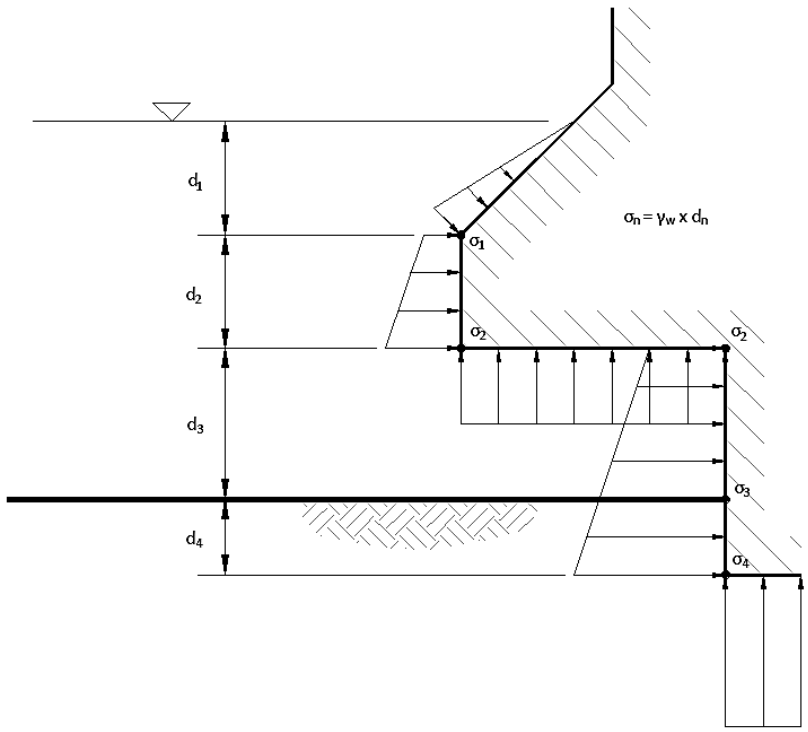


Figure C5.4-1. Example Hydrostatic Pressure Distribution.

C5.4.2.3 Seepage

The state of saturation of the soil and the location of the phreatic surface during and after a flood will dictate the hydrostatic loads applied to the portions of structures below grade. The rate of seepage of floodwaters through soils around and underneath a structure is affected by different factors including ground surface and soil permeability, time of inundation, and the flow path of water through soils. For short-term increases in water depth, such as during coastal storm surge, soils adjacent to or beneath a structure may not become fully saturated during a flood event and designing for full hydrostatic pressures below grade may be overly conservative.

Mitigation of seepage using active stormwater control systems (i.e., sump pumps) can be included at the discretion of the owner and engineer. A seepage analysis can also be used to evaluate the design discharge rate for the purposes of sizing these systems. The reliability of these systems should be evaluated with regards to long-term storage, regular maintenance, and a back-up power supply, to ensure proper operation during flood events.

C5.4.3 Hydrodynamic Loads

Hydrodynamic loads are those loads induced by the flow of water moving above the ground level. They are usually lateral loads caused by the impact of the moving mass of water and the drag forces as the water flows around the obstruction. Hydrodynamic loads are computed by recognized engineering methods. In the coastal high hazard area, the loads from high-velocity currents due to storm surge and overtopping are of particular importance. The *Coastal Engineering Manual* (USACE 2002) is one source of design information regarding hydrodynamic loadings.

C5.4.3.1 Drag Force on Components

This section provides the hydrodynamic drag equation (5.4-4) for individual components of structures (i.e., walls and columns). Each element of the structure exposed to the flood load should be checked for the component drag load. The C_{cx} coefficient accounts for accumulation of debris between vertical elements that effectively increases the tributary width on a vertical element. The C_{cx} coefficient is determined per Section 5.3.9.2. For individual elements both sides of the column or wall are considered to have debris damming.

C5.4.3.2 Drag Force on Lateral Force Resisting System

This section uses the same hydrodynamic drag equation to establish the overall force on the structure's lateral force resisting system. For buildings with solid walls, Equation (5.4-5) is used to determine the drag force on the structure. For open buildings or buildings with breakaway walls the drag force is a summation of the drag on individual components (i.e., columns and walls). In addition, debris damming needs to be considered per Section 5.3.9.2 that will increase the effective tributary width of the element. While for component design all vertical elements need to consider this damming it is not reasonable to consider every column with this increased drag simultaneously thus the requirements of Section 5.3.9.2 limit the number of bays that need to consider damming for the design of the lateral force resisting system.

C5.4.4 Wave Loads

Wave loads result from water waves striking a building or other structure. Design of buildings and other structures subject to wave loads should account for the following loads: waves interacting with a portion of the building or structure; uplift forces caused by waves beneath a building or structure, or portion thereof; wave runup striking a portion of the building or structure; wave-induced drag and inertia forces; and wave-induced scour at the base of a building or structure or its foundation.

The magnitude of wave forces (lb/ft^2) (kN/m^2) acting against buildings or other structures can be many times higher than wind forces under design conditions. Thus, elevating the structure above the wave crest elevation is crucial to the survivability of buildings and other structures. The portion of the elevated structure~~s~~ located within the design stillwater flood depth must be designed for large wave forces.

Wave load calculations using the analytical procedures described in this standard ~~all~~ depend on the initial computation of the wave height and period described in Section 5.3.7.

C5.4.4.1 Wave Loads on Vertical Piles and Columns

Breaking wave loads tend to be significant, particularly on vertical piles or columns. Floods loads primarily exert lateral loading, and slender foundation elements should be designed for these loads both structurally and geotechnically. In general, the lateral load exerted on piles are resisted by soils with depth and will result in larger bending moments compared to assuming fixity at grade. This soil-structure interaction is typically taken into consideration by developing inelastic soil (*p-y*) springs to represent soil resistance at depth along the length of the pile, or by developing an equivalent depth to pile fixity without soil springs. For the latter approach, the depth to the point of fixity shall be determined at the depth that produces the same top of pile displacement as that given by an individual lateral analysis for a given lateral load at the top of pile.

C5.4.4.1.1 Nonbreaking Wave Loads on Vertical Piles and Columns

The maximum net wave force due to a nonbreaking wave is based on a publication by the US Army Corps of Engineers (USACE) (2002). The isolines contained in the figures (Figures VI-5-131 to VI-5-134 of USACE 2002) are used for development of the force coefficient.

While ϕ_m often can be lower than 0.5, the standard has conservatively considered the maximum possible force coefficient for the wave loading formula. If permitted by the Authority Having Jurisdiction, for round piles or round columns, other values of ϕ_m may be used based on Figures VI-5-131 to VI-5-134 of USACE (2002). The wave force should be considered to act at the design stillwater flood elevation; however, Figures VI-5-135 to 138 of USACE (2002) can be used for estimation of the moment generated by the wave forces at the groundline. The moment at the groundline when divided by the net wave force yields the location of the wave force with respect to the groundline.

When W exceeds 1.0, the Morrison equation, which is the basis for USACE (2002) figures, is no longer applicable, in which case the vertical pile or column behaves as a wall. Therefore, the formulae for nonbreaking wave loads on non-elevated vertical walls may be used for wave force determination.

The wave drag coefficient, C_D, varies depending on the Reynold's Number, R_e, according to the following relationship:

$$C_D = \begin{cases} 1.2, \text{ where } R_e < 1 \times 10^5 \\ 1.2 - \frac{R_e - 2\times10^5}{6\times10^5}, \text{where } 1 \times 10^5 < R_e < 4 \times 10^5 \\ 0.7, \text{where } 4 \times 10^5 > R_e \end{cases} \qquad \text{(C5.4-1)}$$

where

R_e = Wave Reynolds number given by:

$$R_e = \frac{u_m D}{\nu} \qquad \text{(C5.4-2)}$$

and where

ν = Fluid kinematic viscosity in ft^2/s (m^2/s). For seawater, ν = 1.21E-5 ft^2/s (ν = 1.12E-6 m^2/s), and for freshwater, ν = 1.08E-5 ft^2/s (ν = 1.00E-6 m^2/s)

u_m = Maximum horizontal velocity in ft/s (m/s) given by

$$u_m = \frac{g\, H_{design}\, T_p}{L} \qquad \text{(C5.4-3)}$$

Typically, for shallow water waves, R_e is greater than 4×10^5. Therefore, the standard incorporates a drag coefficient of 0.7 for round columns or piles.

Research on wave drag coefficients for square or rectangular cylinders is limited. An experiment carried out by Venugopal et al. (2006) indicated that the range for C_D is 2.0 to 4.0 for KC > 3.0, where KC is Keulegan-Carpenter number. KC is generally over 10 for shallow water wind driven waves. Therefore, a drag coefficient of 2.25 has been incorporated in the standard, which is also consistent with the breaking wave formula in the standard.

C5.4.4.2.1 Lateral Nonbreaking Wave Loads on Non-elevated Vertical Walls

Equations for nonbreaking wave loads on vertical walls presented in Section 5.4.4.2.1 are based on the methodology proposed by Goda (1974) and presented in USACE (2002), Goda (2010), and Tomiczek et al. (2019). Equations presented in Section 5.4.4.2.1 are modified for non-elevated vertical walls on a horizontal submerged ground surface subject to normally incident water waves

The original equations were derived for nonbreaking waves acting on a vertical caisson breakwater elevated on a rubble mound with crest elevation at depth *d,* the base of the upright section at depth *h',* and depth offshore of the breakwater at depth *h.* The horizontal pressure distribution is defined

$$\eta^* = 0.75\,(1+\cos(\beta))H_{design} \qquad \text{(C5.4-4)}$$

$$p_1 = \frac{1}{2}(1+cos(\beta))(\alpha_1\lambda_1 + \alpha_2\lambda_2 cos^2(\beta))\rho_w g H_{design} \quad \text{(C5.4-5)}$$

$$p_2 = \begin{cases} \left(1 - \frac{h_c}{\eta^*}\right) p_1 & for\ \eta^* > h_c \\ 0 & for\ \eta^* \le h_c \end{cases} \quad \text{(C5.4-6)}$$

$$p_3 = \frac{p_1}{cosh\,(\frac{2\pi h}{L})} \quad \text{(C5.4-7)}$$

$$p_4 = \alpha_3 p_1 \quad \text{(C5.4-8)}$$

where all previously defined variables are consistent with Section 5.4.4.2.1 and

ρ_w = Density of water in lb s^2/ft^4 (kg/m^3), taken as 1.94 lb s^2/ft^4 (1000 kg/m^3) for fresh water and 1.99 lb s^2/ft^4 (1027 kg/m^3) for seawater, and

p_4 = Pressure in lb/ft^2 (kN/m^2) at the base of the structure.

Wave pressure coefficients α_1, α_2, and α_3 are calculated as

$$\alpha_1 = 0.6 + \frac{1}{2}\left[\frac{4\pi h/L}{sinh\,(\frac{4\pi h}{L})}\right]^2 \quad \text{(C5.4-9)}$$

$$\alpha_2 = min\left\{\frac{h_b - d}{3h_b}\left(\frac{H_{design}}{d}\right)^2, \left(\frac{2d}{H_{design}}\right)\right\} \quad \text{(C5.4-10)}$$

$$\alpha_3 = 1 - \frac{h'}{h}\left[1 - \frac{1}{cosh(\frac{2\pi h}{L})}\right] \quad \text{(C5.4-11)}$$

with h_b defined as the water depth in ft (m) measured at a horizontal distance $5H_s$ seaward of the breakwater. Coefficients λ_1 and λ_2 account for breakwater geometry and are set equal to unity for a standard upright breakwater. For a horizontal submerged ground surface, h_b=d and therefore

α_2=0. In addition, in the absence of a rubble mound $h=h'$ and the pressure at the bed equals the pressure at the base of the vertical wall or structure ($p_3=p_4$).

C5.4.4.2.2 Lateral Breaking Wave Loads on Non-Elevated Vertical Walls

Breaking waves are plunging waves that may impact a vertical wall or other structure with a nearly vertical front, inducing high magnitude, short-duration impulsive pressures (and associated force) on a wall or vertical structure. Breaking waves may also entrap air pockets that result in a double peaked force associated with the wave crest hitting the structure and subsequent air pocket compression. Breaking wave loads on buildings and other structures can be very large, and conditions resulting in frequent wave breaking at vertical walls and other structures should be avoided when possible. The equations proposed by Goda (1974) were modified to account for impulsive breaking wave pressures by Takahashi et al. (1994) and are presented in USACE (2002). Section 5.4.4.2.2 conservatively assumes that the impulsive wave breaking coefficient, α_{IB}, accounting for berm width, wavelength, and water depth is equal to 1.0.

The Goda (1974) equations for nonbreaking waves with modifications for impulsive wave breaking by Takahashi et al. (1994) have been compared to large and small- scale experiments measuring nonbreaking and breaking wave-induced loads on vertical walls and elevated structures. Predicted forces showed good agreement with laboratory measurements (e.g., Wiebe et al. 2014, Tomiczek et al. 2019).

C5.4.4.2.4 Lateral Breaking Wave Loads from Obliquely Incident Waves

Wave loads on vertical walls reach a maximum when the waves are normally incident (i.e., direction of wave approach is perpendicular to the face of the wall with wave crests parallel to the face of the wall). Obliquely incident waves may be identified by a site-specific study. In the absence of a site-specific study and as guidance for designers of coastal buildings or other structures on normally dry land (i.e., flooded only during coastal storm or flood events), it can be assumed that waves will approach the shoreline within a 22.5-degree angle from perpendicular. Therefore, the direction of wave approach relative to a vertical wall will depend upon the orientation of the wall relative to the shoreline.

Wave force equations provided in Sections 5.4.4.2.1 and Section 5.4.4.2.2 may be modified in instances where waves are obliquely incident. The pressures and forces from obliquely incident waves should be calculated by modifying Equations 5.4-8 and 5.4-9 (or 5.4-14). In the equations, $\eta*$ and p_1 (or p_{1B}) should be multiplied by

$$C_\beta = (1/2)\,(1 + \cos(\alpha_H)) \qquad \text{(C5.4-12)}$$

where

C_β = Coefficient accounting for wave directionality, and

α_H = Horizontal angle between the direction of wave approach and the vertical surface.

C5.4.4.2.5 Lateral Wave Loads on Elevated Walls

Many near-coast structures are elevated above the ground on piles or piers and may experience both lateral and uplift forces during extreme storm surge and wave events. The Goda (1974) equations were modified for elevated structures by Wiebe et al. (2014) assuming a linear pressure distribution between either the pressure at the design stillwater flood elevation and zero pressure at elevation η^* (positive air gap) or the pressure at the design stillwater flood elevation and the pressure at the bed (negative air gap). Large and small-scale physical model experiments measured nonbreaking and breaking wave-induced loads on non-elevated and elevated vertical walls, with predicted forces showing good agreement with laboratory measurements (e.g., Wiebe et al. 2014, Tomiczek et al. 2019).

C5.4.4.3 Wave Uplift Forces on Elevated Structures and Non-Elevated Structures with Overhangs

Uplift forces on an elevated structure are a function of the structure's base dimensions, elevation with respect to the design stillwater flood depth, and wave conditions including wave height and period. Uplift forces have been observed to be significantly greater than horizontal loads, especially for small, nonbreaking incident wave heights. Park et al. (2017) measured horizontal and vertical forces on a 1:10 scale physical model structure and found that the maximum uplift force occurred when the structure's base was positioned at the design stillwater flood elevation. Particularly for small waves, the horizontal to vertical force ratio was less than one, indicating that peak vertical forces may exceed peak lateral forces. Regular wave measurements from the same experiments found a range of vertical to horizontal force ratios of 1.2 to 9.7. Bradner et al. (2011) performed 1:5 scale measurements on a bridge highway superstructure and measured vertical forces three to five times those corresponding horizontal forces.

USACE (2002) presents equations for estimating wave uplift forces due to steady and oscillatory currents or wave slamming. The uplift forces due to currents are considered when the width of the structure is small compared to the wavelength (D/L < 0.2), and is calculated as:

$$F_L = C_L A_N \gamma_w (u^2/(2g)) \qquad \text{(C5.4-13)}$$

where

C_L = An empirical lift coefficient,

A_N = Projected area of the solid body normal to the flow direction,

γ_w = Ppecific weight of water, in lb/ft^3 (kg/m^3),

g = Gravitational acceleration, taken as 32.2 ft/s^2 (9.81 m/s^2), and

u = Magnitude of the flow velocity, in ft/s (m/s).

For steady flow situations, the empirical lift coefficient, C_L, is a function of the Reynolds number, the solid body roughness, and the boundary-imposed flow field around the body.

When the base of the structure is located above the design stillwater flood elevation and subjected to oscillatory wave action, the uplift force may be approximated as

$$F_U = C_U A_Z \gamma_w (w^2/(2g)) \tag{C5.4-14}$$

where

C_U = Laboratory defined wave slamming coefficient,

A_Z = Projected area of the solid body in the horizontal plane, and

w = Vertical component of the flow velocity, in ft/s (m/s).

Sarpkaya and Isaacson (1981) noted laboratory measured slamming force coefficients, C_U, ranging from 4.1 to 6.4 for rigidly mounted horizontal circular cylinders.

Wave uplift forces on a partially submerged elevated structure are difficult to compute due to the modification of the flow field by the structure and nonlinear boundary conditions at the water surface. Physical and numerical modeling can provide estimates of wave uplift forces on elevated structures. Equations have been proposed that estimate the total uplift force on offshore platforms and coastal bridges; often equations are a function of one or more of the following: wave height, wave period, water particle velocity, and structural base dimensions (e.g., Kaplan et al. 1995, Suchithra and Koola 1995, Bea et al. 1999, Douglass et al. 2006, AASHTO 2008). Simpler formulae relating the elevation of the base of a structure with respect to the design stillwater flood elevation are proposed by Wang (1970) and Cuomo et al. (2011).

C5.4.5 Debris Impact Loads

Impact loads are those that result from objects that can be picked up in flood events such as logs, cars, small vessels, and, in larger flooding events, shipping containers, barges, and large ships. Section 5.3.9 defines what debris strikes are applicable for each type of building and component (i.e., façade versus primary structure) and this section covers the specification of loads on structures and structural components by debris objects.

C5.4.5.1 Debris Impact Load Determination

Debris impact forces are to be determined for the location of the structure based on the potential debris in the surrounding area that would be expected to reach the site during the flood. Of particular concern are the perimeter structural components oriented perpendicular to the flow direction because they are at the greatest risk of impact and their loss may compromise the ability of the structure to support gravity loads.

The impact forces depend on the impact velocity, which is assumed to be equal to the flow velocity for floating debris. The points of application of the impact force, which is assumed to be a concentrated force, are chosen to give the worst case for shear and moment for each component that is required to be considered within the inundation depth and the corresponding flow velocity (as required in Section 5.3.9.1). Exceptions to this are specified in subsequent sections based on specific debris characteristics.

C5.4.5.1.1 Simplified Debris Impact Load for Passenger Vehicles or Small Vessels

Designing for a conservative, prescriptive load is allowed to replace specific consideration of impact by vehicles, and small vessels which are the debris types required for all structures. The prescriptive load is based on the maximum force generated based on the elastic procedure of Section 5.4.5.1.2. This assumes the maximum design velocity per Section 5.3.6 of 10 ft/s (3 m/s) and a very stiff structural element (24 in. wide by 14 in. thick concrete wall, remaining elastic k = 500 k/in). The result is controlled by the small vessel (41 kips with C_o factor applied) which is very conservative for most applications. This does not account for a more flexible structural element, the stagnation coefficient, the debris depth coefficient or that in most conditions, the design velocity will be much lower than the required maximum. In addition, this load does not account for inelastic behavior the debris element (i.e., crushing).

C5.4.5.1.2 Elastic Debris Impact Loads

Previous versions of this chapter (Section C5.4.5 of ASCE 7-16) have been based principally on an impulse-momentum formulation for rigid-body impact, which requires an assumption of the duration of impact. Conversely, Equation (5.4-20) is based on stress wave propagation in the debris and hence considers the flexibility of the debris and impacted element.

The elastic debris impact equation in Section 5.4.5.1.2 is essentially a reformulation of the impulse momentum approach using the stiffness of the debris and element struck to establish the impact duration. This equation is a simple approach for determining the required impact forces and requires minimal information.

The primary assumption is the debris object remains elastic during impact and strikes the structure longitudinally. The stiffness is typically taken of the debris object assuming the impacted component is completely rigid, however the impacted component stiffness can be significantly less than the debris (in the case of a column or out of plane wall). If desired the lateral stiffness of the component can be included to determine an effective stiffness using Equation (C5.4-15):

$$k_{eff} = 1/(1/k_{debris} + 1/k_{structure}) \quad \text{(C5.4-15)}$$

The debris velocity coefficient (C_R) is a coefficient used in previous editions of this document (ASCE 7-10, C5.4.5) to recognize that in shallow flooding scenarios, bottom drag plays a significant role in the velocity of the debris. Thus, for flow depths of less than 5 ft (1.5 m) the effective debris velocity is reduced from the design velocity.

The value of the debris orientation coefficient, C_o, was derived from the data of Haehnel and Daly (2004), jointly sponsored by ASCE and FEMA. It is the mean plus one standard deviation value of the log debris impact force for trials that included glancing and direct impacts of freely floating logs. Therefore, C_o will remain as 0.80 used in prior versions of Chapter 5 to account for the less likely head-on strike of the object on a wall. The 0.80 factor correlates to an object strike that is approximately 36 degrees between the longitudinal axis of the debris and the flow direction.

The debris velocity stagnation coefficient, C_s, has been added for Chapter 5 based on recent laboratory testing and recognition that a more accurate load characterization was needed when an architectural component is the critical resisting element. C_s is a factor to locally reduce the velocity of the impacting debris based on the effect of water level increasing as it stagnates on the face of a wider building (refer to Figure C5.4-2). This is substantiated by testing as described in Shafiei et al. (2016) and Derschum et al. (2018) which observed that the debris velocity decreased as it approached a structure. This flow stagnation will significantly reduce the velocity of debris strikes to the building near the center. The factor provides a reduction in velocity based on location across the face of the building. The factor is limited to non-load bearing elements on the face of closed buildings (i.e., no flow through) and buildings wide enough to exhibit the effect. In addition, for stagnation to occur, there cannot be any open flow through the structure (i.e., open structure), flow below the structure (i.e., elevated structure), or walls cannot be designed to break away during flood loading. The stagnation effect is typically applicable for façade elements and their back-up framing subjected to the flood loading, not primary gravity structure such as columns located inside the façade. A graphical illustration of the effect is shown in Figure C5.4-2. The perpendicular flow velocity at the sides and the back of a building (relative to the direction of the flow) are significantly smaller than at the front corners. However, unless a site-specific study is performed to establish the flow direction, flow is required to be considered in all directions for coastal sites.

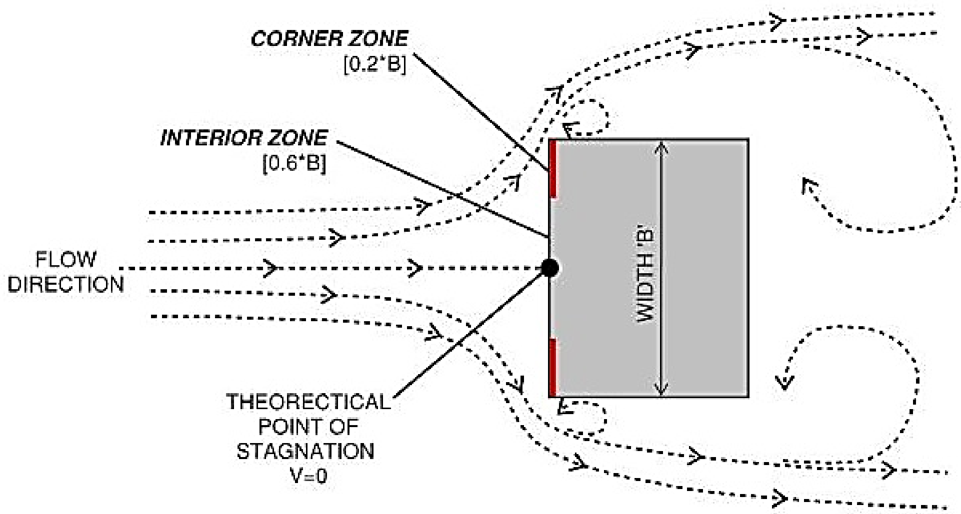


Figure C5.4-2 Flow Stagnation at the Face of the Building.

For some debris/structural elements, the assumption that the debris and structure remain elastic can be extremely conservative, resulting in significantly higher forces than may actually be observed. For most debris strikes, the impacted elements impacted will experience a response in their inelastic range; thus, the element will have a much lower effective stiffness. However, using a cracked or reduced stiffness can be unconservative if the proper analysis is not done. For example, a CMU wall seeing a debris strike will likely yield the reinforcing, crack the tension face of block and possibly see crushing at the compression face at the point of maximum moment. However, at the ends of the walls (in a pinned condition) the walls remain in their uncracked state. Using the work energy approach of Section 5.4.5.1.3, the effective stiffness of walls can be determined considering the changing effective stiffness over the wall height. If the methods of the elastic approach lead to failing elements by a reasonable margin, it is likely a more advanced approach using the structural elements' nonlinear response could be used to justify the elements adequacy. However, if this approach is taken, any permanent deformation or cracking must be evaluated against any dry floodproofing leakage performance, as defined in ASCE 24, of the watertight barrier (if any).

C5.4.5.1.3 Alternative Methods of Debris Impact Analysis

It is permissible to use one of the following nonlinear methods in lieu of the elastic debris impact method of Section 5.4.5.1.2. An equivalent single degree of freedom mass-spring system with a nonlinear stiffness that considers the ductility of the impacted structure for the dynamic analysis is permitted to be used to determine the impact forces. Alternatively, the structural response is permitted to be calculated based on a work-energy method with nonlinear stiffness that incorporates the ductility of the impacted structure. The velocity used for this analysis is VC_RC_s C_o as defined by Section 5.4.5.1.2. Impact loads are to be applied to the structure over areas defined in Section 5.4.5.2 to produce the most critical flexural and shear demands.

Where debris impacts from shipping containers, ships and barges exceed the acceptance criteria of a structural element, it is permitted to accommodate the impact forces through progressive collapse provisions of the recognized literature. Progressive collapse provisions are applied up to the design stillwater flood depth plus the height of the deck of the vessel from the waterline.

C5.4.5.2 Debris Types and Properties

Wood Logs and Poles

The assumptions used to tabulate the debris stiffness for wood log and pole strikes are that the impact is elastic and that the pole/log is longitudinal when it impacts the structural element. That is, in the case of a pole or log, it hits the structure with its butt end rather than broadside. However, it is unlikely that a true longitudinal strike will occur in a turbulent flow over uneven ground. The orientation factor is provided to adjust for a non-longitudinal strike.

A 1,000 lb (4.5 kN) object can be considered a reasonable average for flood-borne debris and is consistent with ASCE 7-98 through ASCE 7-16. This represents a reasonable weight for trees, logs, and other large woody debris that is the most common form of damaging debris. This weight corresponds to a log approximately 30 ft (9.1 m) long and just under 1-ft (0 3-m) in diameter. The 1,000 lb (4.5 kN) object also represents a reasonable weight for other types of debris ranging from small ice floes to boulders to man-made objects.

However, design professionals may wish to consider regional or local conditions before the final debris weight is selected. In riverine floodplains, large woody debris (trees and logs) predominates, with weights typically ranging from 1,000 lb (4.5 kN) to 2,000 lb (9.0 kN). In the Pacific Northwest, larger tree and log sizes suggest a typical 4,000 lb (18.0 kN) debris weight. Debris weights in riverine areas subject to floating ice typically range from 1,000 lb (4.5 kN) to 4,000 lb (18.0 kN).

The elastic stiffness of the log can be established using structural mechanics (AE/L) for a Douglas fir larch pole 30 ft (10 m) long, measuring approximately 12 in. (0.3 m) in diameter. A stiffness of 350 to 550 kips/in (4,200,000 to 6,600,000 lb/ft) can be derived using structural properties from the National Design Specification (NDS). Lab testing sponsored by the USACE (refer to Haehnel and Daly 2002) showed the effective stiffness of a log is likely a fraction of the elastic stiffness calculated above. This testing set the upper bound of a straight-on impact at 2.4 MN/m, which correlates to a 14 kips/in (168,000 lb/ft) effective stiffness. However, more recent research (Piran Aghl et al. 2014) indicates a higher stiffness, consistent with the calculated elastic stiffness, which is used as the minimum requirement.

Vehicles

Passenger vehicles are ubiquitous, float, and are easily transported. This assumes that impact occurs, as long as the inundation depth is sufficient to float the vehicle, which is deemed to be 3 ft (0.91 m). Research by NCAC (2011, 2012) describes an experimental and numerical analysis of the frontal crash impact of a 2,400 lb (10.7 kN) subcompact passenger vehicle traveling at 35 mph (15.6 m/s) against a wall. Based on the results therein, the initial stiffness of the vehicle was estimated to be 5,700 lb/in (998 kN/m), which has been rounded up to 6 kips/in (72,000 lb/ft) (1,051kN/m). While it is acknowledged that larger, heavier, and stiffer vehicles are likely to be within the floodplain, the lighter compact style cars are most likely to float freely and are considered in this standard as a minimum. The designer may choose to evaluate a larger debris impact but the smaller car more likely to float is established as the minimum requirement.

Small Vessels

In a flood event. there is likely to be a number of small vessels (typically 16 to 30 ft) (4.9 to 9.1 m) in length in the flood area. These types of vessels are typically of aluminum, wood or fiberglass construction and are not ocean-going vessels. These debris objects are typically limited to vessels of 2,500 lbs (11.1 kN) due to their size and construction type thus this is established as a minimum stiffness for design. The stiffness required for design is the axial stiffness of the vessel (*AE/L*), with the largest stiffness coming from a straight-on bow impact.

These vessels are typically 6 to 8 ft (1.8 to 3.6 m), in width resulting in a maximum hull cross sectional length of approximately 100 to 150 in. (significantly less at the front at the point of impact). Thickness varies from 0.25 to 0.75 in., depending on location. Based on the size and length of these vessels and typical construction materials (elastic modulus approximately 10,000 ksi for aluminum) a minimum effective axial stiffness for a theoretical straight-on impact engaging the entire cross section could exceed 500 kip/in. However, the likelihood of a straight-on impact aligning with the point of the bow on a perpendicular surface is highly unlikely. Therefore, the effective stiffness of an offset impact on the bow region is considered for the minimum design stiffness. Based on a sample of small vessels considered with parametric studies showing slightly off-center impacts in the bow region an effective stiffness of 30 kips/in [360,000 lb/ft (5,254 kN/m)] is established as the minimum for design. This is significantly lower than the straight-on stiffness due to bending in the sidewalls of the vessel being considered. There may be vessels in this size range with effective stiffness higher than this value, however this was deemed as a reasonable minimum threshold for design. A designer should always consider local conditions that may result in a higher mass or effective stiffness.

Shipping Containers

Shipping containers, like cars, are likely to float in a flood event. These containers have been shown to reach the full water velocity and have significant mass. Test results (Piran Aghl et al. 2014) have shown that the mass of container contents does not significantly affect the impact force as long as the contents are not rigidly attached to the structural frame. Therefore, for shipping containers, the mass of the container without contents is used in Equation (5.4-20). Shipping containers are standardized in terms of length, height, and width, but weight and structural details can vary somewhat by manufacturer. The values for weight and stiffness provided in Table 5.4-4 are considered to be reasonable approximations for most standard ISO shipping containers. Hence, these numbers converted to mass can be used directly in Equation (5.4-20) for m_d. The stiffness values are based on EA/L, where E is the modulus of elasticity of steel, A is the cross-sectional area of one bottom rail of the container, and L is the length of the rail, not including any cast end blocks.

Equation (5.4-20) does not contain any factor to account for an increase in force caused by the fluid flow being affected by the sudden stoppage of the debris object, which some other formulations include. For longitudinal impact of a log, such an increase in force is not expected to be significant. Testing of scale-model shipping containers also showed that for longitudinal impacts, the impact force was not significantly affected by the fluid (Ko et al. 2015). The force from Equation (5.4-20) is considered to be sufficiently conservative to allow the transient fluid "added mass" effect to be ignored.

The maximum impact force of a 40 ft (12.2 m) shipping container on a Risk Category III structure of 224 kips (996 kN) can be calculated using the maximum flow velocity (for Risk Category III structures) and a rigid structure impact using the elastic impact equation. Research by Piran Aghl et al. (2014) indicates a maximum force of 220 kip (980 kN) at 12.5 ft/s (3.8 m/s) which is a slightly lower velocity than the required maximum velocity for flood loading of Risk Category III structures, which is in line with the calculated impact force.

Ships and Barges

Vessels that weigh between 2,500 lb (11.1 kN) and 88,000 lb (391 kN) are required to be considered when they are present (refer to Section 5.3.9.1.1). The vessels considered in this section are often ocean going vessels or large capacity barges greater than 30 ft (9.1 m) in length.

When a structural element cannot be economically designed for the forces required by the debris impact, it is permissible to let that structural element fail and use a progressive collapse analysis to show the structure will not collapse. This approach is allowed for the medium-large debris types (ships/barges and shipping containers) to reduce the design effects of a debris strike. The common coastal construction type on the Gulf and East Coast is an elevated structure supported by a large number of wood piles (often diameter of 12 to 16 in.) and small spacing. These piles often have a very limited lateral capacity limited by geotechnical considerations and it would not be feasible to design these piles for debris strikes. As an economic compromise the exception allows these piles to fail and a transfer beam to be designed considering the failure of a pile below to transfer the load.

The exception considers the typical construction type of wood piles supporting an elevated structure on the Gulf and East Coast. These types of wood piles have minimal lateral capacity in soil and may not be able to resist larger impact forces individually. As a tradeoff, these piles may fail individually, and progressive collapse provisions may be used to evaluate the surrounding structure. Piles of other materials, such as steel and concrete, may also have limited individual lateral capacity dependent on the geotechnical conditions in which they are installed.

C5.4.5.3 Extraordinary Debris Impact

Extraordinary debris impacts, defined as impact by 88,000 lb (391 kN) or larger marine vessels, should be considered for Risk Category IV buildings and structures that are in the debris hazard impact region of a port or harbor, as defined in Section 5.3.9.1.2 and for which the design flood depth is 12 ft (3.7 m) or larger.

The size vessel to be used depends on the most probable size vessel typically present at the port or harbor. The harbormaster or port authority can be consulted to determine typical vessel sizes, ballasted drafts, and weight displacement under ballasted draft. Typical vessel sizes are also provided in PIANC (2014).

The nominal impact force is calculated with Equation (5.4-20), with the assumption that the vessel stiffness is much larger than the structural member stiffness, thus the transverse structural member stiffness is to be used. The calculated force may be larger than any economical capacity of typical structural elements. In that case the member may be assumed to have failed, in which case progressive collapse should be prevented for these important structures. The provisions require that progressive collapse be checked at all levels where the extraordinary debris strike could occur (with large ships this could be well above the Design Stillwater Flood Elevation).

C5.4.5.4 Debris Impact Load Redistribution

When it is not practical to design an element to take the large forces (often a shipping container, large ship or barge), the use of progressive collapse prevention is an alternative to mitigate the debris strike. An accepted procedure for designing for progressive collapse is the ‘alternate load path’ procedure given by *Design of Structures to Resist Progressive Collapse* (DOD 2013). This procedure requires the structural system to be able to carry the gravity loads of the building while sustaining a loss of a vertical load carrying element. All gravity load carrying elements that are required to sustain debris strikes need to be independently considered as lost for this analysis.

C5.5 FLOOD LOAD CASES

The effects of different flood loading conditions can often occur simultaneously. As a result, Section 5.5 requires several flood load cases to be considered in combination as the flood load effect (F_a) that is combined with other forces in Chapter 2.

Cases 1 and 2 are required for analysis of both individual elements (e.g., columns or facades) and the load-resisting systems (i.e., gravity system and lateral system). Structures allowing passage of flood water through unenclosed spaces (e.g., building perimeter not enclosed by cladding or foundations walls) supported on columns, walls, or piles will not experience wave loads on all exposed foundation elements simultaneously. A more accurate representation of wave load behaviors for the stability of the structure would be to apply wave loads to a single row of columns (i.e., the front row), acting concurrently with hydrodynamic drag load applied to all the other rows of columns. In many cases, the effective hydrostatic load is zero as the load is uniform on all faces of the element and cancels out.

C5.5.1 Stability Against Uplift

Structures must be designed to resist flotation due to upward hydrostatic buoyancy. However, since this is a stability check, it is governed by flood load factors that are different from those of Chapter 2. In general, an uplift failure produces a temporary lifting of a structure as a whole, or portion thereof.

The calculation for resistance to uplift in Equation (5.5-1) uses a load factor of 0.9 for dead load where only the self-weight component of the total dead load is included for this stability check. The 0.9 self-weight load factor differs from the 0.6 dead load factor typical in Chapter 2 ASD load combinations, which are traditionally used for stability checks under other load sources, such as wind uplift. This larger load factor allows for a greater percentage of the actual dead load expected to be present during a flood event to be used for resistance to buoyant uplift while simultaneously disallowing consideration of superimposed loads that may not be present during an uplift event. The supplemental resistance term is intended to capture piles or other foundation elements with tension capacity if properly developed throughout the structural frame. Resistance can be found from side friction of embedded below grade elements as they attempt to lift. Resistance may also be provided by overburden that will remain in place over shallow foundation elements during flood events. Appropriate conservatism should be accounted for in the geotechnical parameters for the flooded condition. If supplemental uplift resistance from these additional elements is being taken into consideration, the engineer should ensure that the connections from these elements to the structure are capable of transmitting the tensile load, and that the load path from the point of buoyant load application to the point of uplift resistance is complete and designed accordingly. All individual elements that are part of the complete uplift resistance load path should be checked with their appropriate material-specific factors of safety.

It is important to evaluate uplift resistance on the overall structure and also at portions of the overall structure with non-uniform self-weight or uplift resistance, and the interface between these different portions of the overall structure. For instance, if a multi-story structure has a one-story extension with an integral basement, the effects of buoyancy should be evaluated on the entire structure considering the differing self-weight and uplift resistances and check the interface for differential shear, bending, and displacement to assure proper performance.

The 0.6 factor on the wind uplift load in this stability check is intended to convert the wind load from strength design levels to allowable stress design levels.

C5.5.2 Stability Against Sliding

Similar to evaluating stability against uplift, the load factors in Chapter 2 must be modified to evaluate sliding stability. Since sliding resistance of structures typically relies on dead weight to engage friction at the base of foundations, buoyancy effects can severely impact the normal force and corresponding sliding resistance. This reduction in the net downward force used to determine frictional resistance shall include reduction due to buoyancy as indicated in the load combinations provided. Frictional sliding coefficients on soils vary depending on the roughness between interfacing materials, (e.g., concrete and steel will have different coefficients on sands) and vary in general depending on the soil gradation. Multiple sources exist for sliding coefficient estimates and should be selected at the discretion of the designer. One available resource for sliding friction coefficients is NAVFAC (1986), though it is specific to retaining walls. The designer should also consider the potential for preferential slip planes, such as smooth geotextiles placed at subgrade or weak seams of soil beneath bearing strata, which may dictate the sliding stability of the structure. Effects of these slip planes can be evaluated through a geotechnical slope stability analysis that appropriately models shear strength parameters at these planes.

The term F_{BR} that is calculated for evaluating buoyancy resistance is used in Equation (5.5-2) for consistency in the treatment of dead weight in a flooded state. Additional resistance to sliding may be achieved by passive earth pressures parallel to direction being analyzed and friction on the sides of structures parallel to the direction being analyzed. Supplemental permanent or temporary structural bracing elements may also be counted upon for resistance if the designer knows they will remain in place and be activated during the design flood event. The designer should be aware that if passive soil pressures are relied upon for sliding resistance, the structure may be expected to slide slightly to mobilize passive forces.

Sliding stability as it is discussed in this section is specific to conditions expected during a flood event, which will differ depending on the type of flooding experienced. Coastal flooding is typically caused by storm surge events that can be accompanied by strong winds. Riverine or lake flooding are not necessarily associated with high wind speed events. The designer should also be aware that lateral resistance derived from different sources such as piles, sliding friction, passive resistance, and structural bracing all mobilize at different strains. The effects of soil-structure interaction should be evaluated by a geotechnical engineer to provide the desired performance of the building.

The 0.6 factors on the wind uplift and lateral loads in this stability check are intended to convert the wind load from strength design levels to allowable stress design levels.

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